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Journal of the
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GENERALITIES ON COASTAL PROCESSES AND PROTECTION

J. B. Schijf[1]

ABSTRACT

The engineering basis of coastal protection is discussed in general with some emphasis on experience on the Dutch seacoast. The paper supports a shift in the old philosophy of protection by groins and seawalls to a new philosophy of protection by sand fill.

Never before in history there has been such a wide-spread interest in the coast on the part of the engineer, both as research worker and as constructionist, as now, and it may safely be said that during the last quarter of a century the understanding of coastal processes has advanced by leaps and bounds. It is a rather humiliating thought for mankind that a main reason for this advance is formed by the war, though an optimist might perhaps take this as an example of the usefulness of wars.

Another reason, and one for which mankind cannot be very proud, has been the fact that human interference with coastal processes (although always motivated by entirely laudable considerations) has become so destructive of late, that it is absolutely imperative to do something about it. Still another reason, and a more fortunate one this time, is the advance in observational technique and in the physical and mathematical methods necessary for a sound qualitative and quantitative treatment of the problems encountered.

Still, the more learned about coastal processes, the more it is realized how little we know yet. Also the impression is gained that much of the knowledge has not penetrated sufficiently to the minds of many people who have or should have responsibility for the maintenance of the coasts and also that there is some danger of engineers becoming so deeply entangled in a multitude of detailed problems—all of them entirely worthy of attention—that they are apt to lose sight of a few simple truths.

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1. Chf., Central Research Div., the Rijkswaterstaat, 's-Gravenhage.

By these considerations, this paper offers some general remarks, even though most if not all of them have been expressed before, e.g. by R. Eaton and M. A. Mason at earlier Coastal Eng. Conferences and other occasions, and also in some unpublished Dutch reports.

The first of them is the necessity to distinguish between the movement of sand perpendicular to and along the coast. Many of the difficulties and mishaps experienced in coastal defence work, are attributable to a lack of appreciation of this distinction. First consider the behaviour of straight beaches and shores under the impact of regular waves approaching straight on and in the absence of other agents for a lateral drift. Very valuable studies have been made on the equilibrium profiles associated with this mechanism. Of course the conception of equilibrium in this connection has a relative significance, because at no time is the profile actually at rest. The materials from which it is built are not in a stable position, but they are continuously or intermittently in motion. The term "equilibrium" should be understood in this sense: that during regular intervals the profile repeats approximately the same sequence of identical shapes and, as long as the waves are absolutely regular, these shapes do not diverge greatly. But of course outside of the laboratory such a regular wave-action does not exist; in nature at best the wave-field has a constant spectrum. In that case engineers may (and do) employ the term equilibrium profile, because still a certain sequence—be it a slightly more varied one—will be regularly repeated. But the deviations between the extremes will be greater and they cover longer periods, associated with the character of the wave spectrum. Such a state of relative equilibrium will always remain a temporary affair, that—for instance on a tide-less coast under the sway of a regular ocean-swell—might last during an appreciable period, but on other coasts—where the waterlevel rises and falls and windwaves of less venerable age dominate—it will never really materialize.

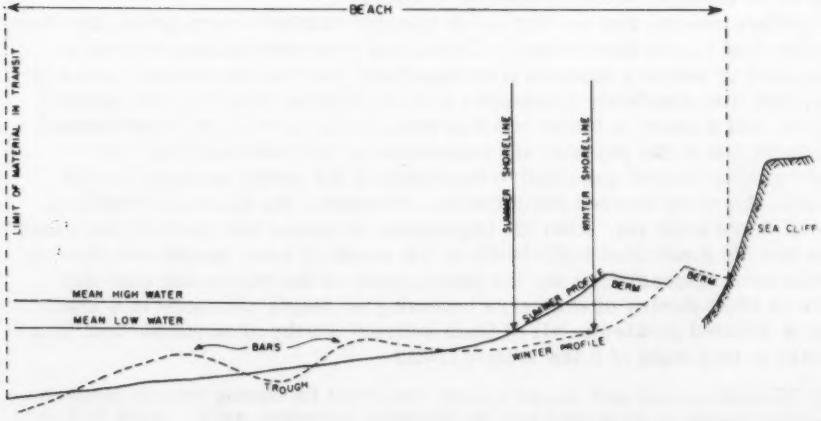

Fig. 1. Seasonal Variations of a Beach Profile.
(Bascom, Coastal Engineering 1957)

But again the limits of conception of equilibrium may, for instance, be extended to cover a succession of tidal cycles, and—still wider—a succession of recurring cycles of identical or at least grossly identical seaconditions. This type of recurring wave cycle brings the seasonal variations which on certain coasts show a fairly true repeatability. (See Fig. 1) Of course the longer the period of the cycle and the greater the variations in seaconditions are, the larger also are the deviations between the extreme profiles of the equilibrium cycle; in the seasonal case, this difference is between the summer and winter profiles.(1)

On other coasts—particularly of such as the North Sea and similar seas—there are even important variations from one year to another.

The effect of an individual winter season on these coasts is fairly well characterized by the number and severity of the gales occurring during that winter.

Their effect not only depends on the intensity of the wave action they engender, but perhaps mainly on the rise in sea level that accompanies them. Thereby they erode not only the beach, but often even directly the front slope of the dunes. In this way during a single gale like that of 1 February 1953 in the North Sea a section of the dune of some 20 to 30 metres can be swept away and it may take a series of years with relatively quiet winters to build it up again. (See Fig. 2) Still, in the absence of lateral drift and provided no material gets lost in some way or other, this may eventually happen (on our coast it usually does not). Therefore even in conditions like these we may still employ the conception of equilibrium, although the deviations between the extreme profiles may be considerable and a more or less periodic cycle may not be in evidence. There is still an equilibrium in this sense that no lasting erosion takes place and that more or less identical profiles appear again and again, although they are by no means stable.

For the maintenance of such an equilibrium the natural forces require two things: space to play in and a stock of sand to play with. The space required is limited by the two extreme positions of the beach and dunefront, the

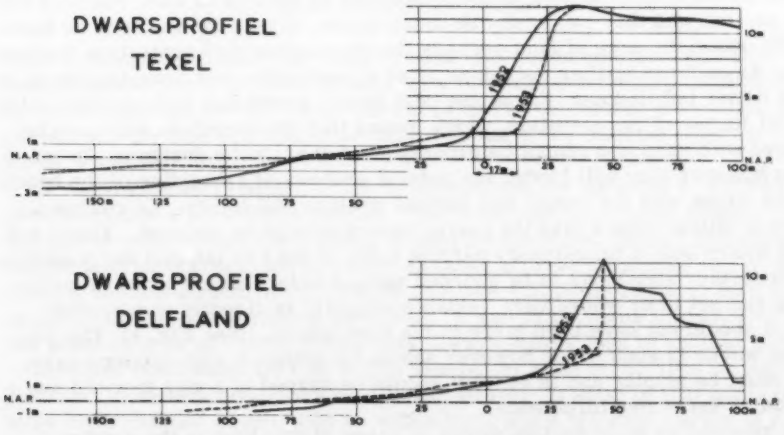

Fig. 2. Dune Erosion after 1953 Gale.

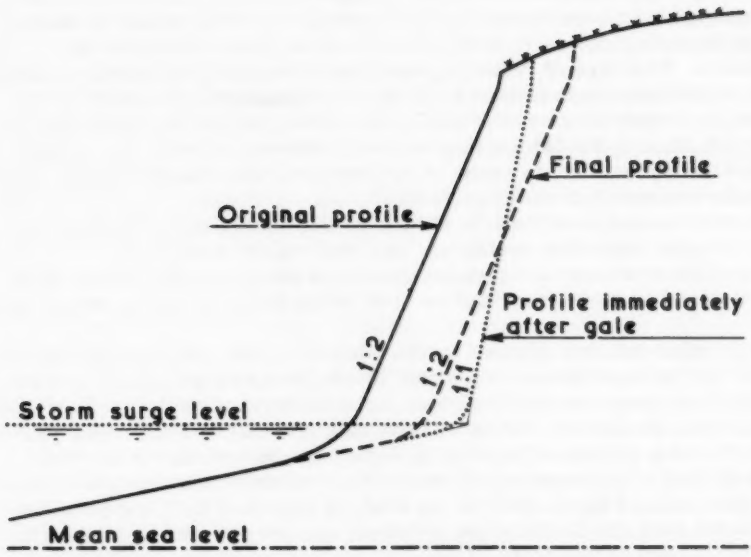

Fig. 3. Relation between Storm Surge Level and Dune Erosion.

stock of sand is being shifted to and fro continuously within this zone. (See Fig. 3) The only thing to guard against is not to hinder the play of the natural forces. Construction should be kept out of this zone and the buffer stock of sand should be left alone.

It is clear that in the absence of a resulting lateral transport, groins or other structures designed for or having the effect of impeding such transport have no meaning. On the other hand they are also rather harmless.

But constructions aiming at the prevention of shifting of sand seaward are also useless and they can easily do much harm. It may for instance be hard to with-stand the urge of (and perhaps the clamouring for) protecting the foot of the dunes from further recession after a heavy gale, but nevertheless it is much better left undone. For at the best such a protection will not hinder the natural forces of recuperation, which means that the structure will soon be covered with sand and cannot be proudly pointed at by the designer. In many cases however they will hinder the natural process of rebuilding of the beach and the dunes with the result that further erosion will follow. Of course a policy of letting nature take its course cannot always be pursued. There are cases where only a dangerously narrow ridge of dune is left and the possibility of a further attack has to be guarded against before a new phase of building up has set in or where very valuable property is directly endangered (only it should not have been there in the first place). (See Fig. 4) The protective works in such cases however should be designed with extreme care. They must be sloping and in general should be shaped in a way to avoid reflection or excessive turbulence.

An alternative to protective works in cases of this kind is the artificial supply of sand to the endangered spot. The sand may be placed either in front

Fig. 4. Buildings Erected Too Far Seaward Necessitate Erosion Protection.

of or (preferably) behind the ridge of dune that needs reinforcing. This procedure does not carry the same risk as a protective structure as regards worsening the erosive action of the waves and is therefore, if feasible, to be preferred. By this procedure we can do no harm, by building fixed structures, we easily may.

It might also be tempting to try to consolidate the dunefoot in a forward position, after a period of advance. This would not be coastal defense proper, but rather come under the heading of land reclamation. Such a temptation should also be disregarded however, because no real advantage can be gained from it and the structures for such a purpose will do more harm than good, in particular on the adjoining stretches. It has to be borne in mind that along the coast gain in one place practically always means loss in another place.

Another conclusion is, that within this buffer zone structures or buildings for other purposes should not be allowed either. In spite of that many otherwise clear-headed people with an enthusiasm and a tenacity worthy of a far better cause, strive to encumber the beach and the front and crest of the dunes with promenades, hotels, and other buildings, or, another possibility,

Fig. 5. Remnants of the "Atlantic Wall" after Erosion.

with gun emplacements and pillboxes. In too many cases the results are not only disastrous to the owners (See Fig. 5) but also dangerous for the coast. When opposing such projects one is confronted with the difficulty to define the width of the buffer zone to be kept clear, in particular on a coast where the fluctuations extend over a number of years and are fairly large and irregular.

From recent studies in the Netherlands it appears that the erosion caused by a heavy gale, is closely related to the mean sea level accompaning the gale. The dune-foot will recede approximately to the point of intersection of this level with the line continuing the slope of the higher part of the beach. Afterwards the crest-line of the dune will recede still further because—in the conditions prevailing in the Netherlands—the dune front will initially assume a slope of 1 in 1, but thereafter gradually adjust itself to an equilibrium slope of 1 in 2. (See Fig. 3) Application of this rule enables, by means of choosing a design—gale or a succession of several ones—taking into account the frequency distribution of gales or rather of storm-surge-levels to arrive at the fixation of a limiting line for buildings, etc.

Thus far it has been assumed that no material is lost from or supplied to the coast. Even on a stretch of coast without any (resulting) littoral drift this assumption is not necessarily valid.

In particular when there is deep water fairly close inshore, sand may be lost by dispersion, that is to say by being carried seaward during a phase of erosion to a depth where it is out of reach of the waves during the following phase of restoration. On the other hand when shallows are found offshore, sand may be carried to the coast. In either of these cases a slow but sure general retreat or advance of the entire zone, in which the coastal processes are enacted, will be superimposed on the fluctuations of the profile. A similar effect will be produced by a slow rise, or fall, of the mean sea level.

When it becomes necessary to counteract a slow but steady erosion of this nature, it is again obvious that the building of groins will be futile. In this case a protection of the dune-foot is also useless; it can at best temporarily slow down the process, but in the long run it will aggravate the situation.

Short of protecting the entire stretch of shore in question by a continuous breakwater or a series of breakwaters—a very unattractive proposition, the only effective remedy is an artificial supply of sand to the beach, which in this case will have to be more or less uniformly distributed along its length.

The preceding discussion has considered a stretch of coast without any littoral drift. Of course this is rather an academic proposition, because such stretches are very rare and even if the net drift is zero, this will be the result of alternating periods of drift in one or the other direction.

On a coast with a littoral drift however the considerations set forth still hold good. The only difference is that if the littoral drift is not in equilibrium, just as in the case of a permanent supply from or loss to off-shore regions, a general advance or retreat will be superimposed on the fluctuations of the profile.

Permanent displacement of the coast, either advance or recession is generally the result of a decrease or an increase of the rate of littoral drift in the sense of its propagation. As a rule, engineer are not concerned about an advancing coast-line—barring the consideration that the advance may be gained at the cost of depletion somewhere nearby—we will here limit ourselves in the first place to the case of an increasing rate of drift. On a coast where the littoral drift is mainly brought about by wave action, this increasing rate of drift means that the three elements (wave energy, effective angle of incidence of the waves, and supply of sand) are not in harmony. This is necessarily a state of incomplete adjustment, because if the same conditions remain in control long enough, an equilibrium will always be established. It follows that either an equilibrium has not yet been obtained or that by some agency an existing state of equilibrium has been disturbed, that is to say that at least one of the three elements mentioned has been modified.

It also follows that in general such a stretch of coast cannot be straight, but will present a convex curve.

Several empirical formulae have been proposed more or less tentatively for linking the rate of transport to the wave-energy and the angle of incidence. By imposing appropriate sets of boundary-conditions and with application of some simplifying assumption they enable to derive mathematically certain forms of coastline. Several authors, notably Pelnard-Considère[2] and Per Bruun[3] have shown remarkable results of this kind. Leaving these mathematics aside for this occasion, it will be observed in passing that the notion of equilibrium has been used by Bruun in a different sense than in this paper.

In dealing with an eroding coast, the engineer must ascertain in the first place, that it is really, that is to say permanently, eroding. In many cases

the littoral drift has an undulating character, that is to say the material moves in humps, as, for instance, Bruun(4) has shown. In that case periods of advance will alternate with periods of recession, so that even generally eroding stretches of coast may show an advance for a while and advancing stretches a retreat. Such a temporary retreat should not be mistaken as a symptom of a permanent erosion. It may be just another element to be included in a still wider conception of equilibrium, contributing to the magnitude of the fluctuations.

Again, of course, as long as it is not absolutely necessary because of threatening danger to the sea defence, the wise course is not to take measures against it.

If it should become inavoidable to interfere, this should be done by way of an artificial supply of sand or as a direct protection of the dune-foot and certainly not by groins or other structures impeding the littoral drift, because thereby the supply of sand to the downdrift stretch would be decreased with consequences possibly far worse than the danger to be averted.

As has been said, a real, that is a permanent, erosion is a result of a relative lack of supply of sand at the updrift end of the stretch in question.

In some cases this can be brought about by natural causes, for instance a decreasing discharge of bedload by a river as a result of modified conditions in its watershed, or because in a deltaic region the distribution of the bedload over different branches has undergone a modification.

The cause may have nothing to do with the supply of sand itself, but, due to a gradual climatic change, the wave-energy or the effective angle of incidence may be varying in such a way that the capacity of the drift mechanism has outgrown the supply. Also tectonic upheavals may be at the origin. But in the great majority of erosion problems the world has to cope with today, it is not nature but mankind that bears the responsibility.

Mankind harnesses rivers and bottles up their bedload, thereby depriving the coast of its lawful tribute; mankind builds breakwaters and harbour moles, and regulates rivermouths and inlets, thereby barring the littoral drift; mankind dredges huge quantities of sand and dumps them in the sea or uses them for reclamation and so robs the coast; mankind inflicts unnecessary or badly designed works of protection along the coasts thereby in many cases creating excessive turbulence and increasing the eroding action of the waves. And while on the whole it is not possible to control the forces of nature working on the coast, the engineer should at least try to control the human forces.

The first thing of course is to refrain from all activity that is not strictly necessary. On a former occasion, Mason has referred to a certain moronic ingenuity that is sometimes let loose on the coast. These things of course should not happen and part of our task is to see that they do not happen. There remain many other manifestations of human activity that are necessary. Rivers sometimes have to be regulated, harbour moles have to be constructed and the depths in fairways for navigation have to be maintained. But every time the engineer should realize what the repercussions of this work on the coast will be and try, to the best of existing knowledge, to minimize them as far as they are harmful (as they generally are).

Even then, in many cases, the results of engineering activities will be erosion of downdrift stretches of coast and counter-measures must be taken. Of course the first step in every case will be to examine the cause of the erosion, and to determine if the cause of the erosion can be remedied. In

some cases that will be found possible, for instance if the littoral drift is cut off by harbour moles or jetties, there is the possibility of bypassing the drift. Or if the natural supply is impoverished by dredging, the spoils can be refed to the coast. Generally speaking, the balance between capacity of drift mechanism and supply can always be restored by bringing the supply up to the required amount.

A second possibility of course is reducing the drift mechanism and the established manner of doing this is the construction of groins. Just how groins should be arranged and constructed has been, and apparently will continue to be for some time, a subject of much study and discussion. Up till now it has largely been a matter of tradition and experience, in which, apart from the function of the groins as drift-reducing elements, the construction technique and the maintenance problem have played an important part. The engineer will be able to tackle the subject more scientifically when a better understanding of the driftmechanism itself has been acquired. This understanding will no doubt show that there is no such thing as a universal solution to the problem, but that the shape and the arrangement of groins have to be varied according to the local conditions. At any rate, they should be groins and not jetties or breakwaters in disguise and they should be as low as is compatible with their reducing effect on the drift. Groins that emerge to an imposing height above the beach cut off the windblown drift and so prevent the formation, development and restoration of dunes. Also of course they should, like structures protecting a dune-foot, not give rise to reflection of waves or excessive turbulence.

Groins may improve the situation along the stretch of coast, where they are situated and updrift, but downdrift they necessarily aggravate the deficiency in supply of drift material.

A great deal of thought and energy has been devoted by a number of people to the arrangement of the downdrift termination of a group of groins so as to avoid or reduce the bad effect on the downdrift stretch. It is of course an illusion that these endeavours should ever be successful, because an accelerated erosion downdrift of a groin system is inevitable if the system serves its purpose well. At best some local effects, such as undercutting or outflanking of the end groin, may be prevented, but the overall result on the further coast will remain. It will therefore be found that after groins have been introduced on a particular stretch of coast, it will become necessary to extend the system downdrift until the end of the physiographic unit has been reached, unless of course a recession can be tolerated.

A number of factors may enter in the decision if artificial supply of sand or the construction of groins should be chosen to combat an erosion of this nature. There always is of course the economic point of view. Generally speaking, on short stretches this factor will be in favour of groins, on long stretches artificial supply will be the cheapest solution. In Dutch conditions the balance will be at a length of the order of 1 km for every 100,000m^3 annual supply required, provided that suitable sand can be found in the vicinity. The type and manner of construction of groins to be chosen, the availability and price of materials on one hand, the availability and cost of transport and conveying of suitable sand on the other are among the factors that have their influence on this balance.

A really exhaustive comparative evaluation of costs is possible only if one has to do with a really limited physiographic unit. If the downdrift boundary

is not well-defined, the influence of the groins may be felt even beyond it and may be difficult to assess. On the other hand the provisioning with an adequate supply of sand continues its benefit along the entire coast downdrift from the location of the supply as far as the drift mechanism extends its influence.

An important point in favour of artificial supply is the fact that it does not entail a permanent commitment. It can do no harm. It can be experimentally adjusted to the requirements of the coast and it can even be abandoned and be replaced by a groin system if later considerations should make this appear preferable after all. Groins on the other hand, once constructed, are there to stay and to be maintained. Badly constructed they constitute a menace. Though it is not impossible to modify, lengthen or shorten them, should the original dimensions not work out satisfactorily, this means again a difficult and costly undertaking.

Therefore other considerations being equal—or perhaps even slightly unequal—it would appear that artificial sand supply is the more natural and more attractive solution.

Evidently, the choice is not always open. Many coasts, especially in old Europe, are by no means virginal. The heritage of past works must be dealt with. As bypassing—or any system of artificial supply—was unknown to them—as far presently avaible records show, it was suggested for the first time at the Navigation Congress at Brussels in 1935;—prior to that, the universal remedy was building groins. It has been left to the U.S.A. as a younger country, neither blessed nor handicapped by an ancient tradition in coastal engineering, to introduce artificial supply as a recognized remedy for coastal ailments and to explore its possibilities and its requirements. It appears most probable that it will become the general procedure in coastal protection, to be replaced by groins or other methods only in a dwindling number of special cases.

As a special class of coastal defence problems an additional factor has to be considered; this is in cases where currents other than wave-induced longshore currents play an important part. This is in general the case in the vicinity of estuaries and inlets. In these regions we find continually shifting tidal channels and they have a major influence on the shore formations. Therefore a second distinction has to be made, just as important as the distinction between cases with or without a resulting littoral drift.

When a stretch of shore is endangered because a shifting channel approaches, none of the measures cited before can be efficacious. As the transporting capacity of a tidal channel easily exceeds many times the capacity of a drift mechanism due to waves, artificial supply can hardly be considered an economic proposition. Groins might have a slight effect, but only if the channel is a very minor one.

A structure designed as an impediment to wave-induced drift, cannot serve to control the flow in a tidal channel that may have depths of 10 to 20 m. or even more. What is needed here is either a longitudinal jetty or a system of transverse jetties, so constructed that they are safe against undercutting and outflanking. When because of the nearness of the channel to the shore, the beach has become too narrow and the dunes have not sufficient width to be safe, it may become necessary in addition to protect the foot of the dunes directly.

As soon as the lateral progress of the channel is being checked by the jetties, it will deepen and scour around their base. Eventually the channel

may become considerably deeper than it was as a free channel and thereby necessitate a strengthening of the base of the structures. If transverse jetties are used, they should have well-rounded heads and be spaced not too far apart in order to limit the scour. Even then they will require constant attention and frequent maintenance.

As long as they are left to themselves, the channels leading to and from tidal inlet usually shift about in a more or less well-ordered and roughly periodic pattern. An individual channel may swing out over quite a distance and then swing back or, more usual, gradually shoal and be replaced by a newly formed channel. The period may be quite long, in some cases fifty years or more. The engineer should always be very watchful before starting any construction interfering with these movements and take good care not to obstruct a channel when it is near the end of its swing, unless it becomes absolutely necessary to safeguard really valuable interests. It should as long as possible be given the chance of wandering back again or dying down, because once an obstruction has been put in its way, and the channel has bitten into it, the natural rhythm will be disturbed and the channel will dig itself in in all meanings of the term.

In some cases it will be possible to help or accelerate a natural development, tending to bring relief to the threatened spot, by a carefully planned dredging campaign. Dredging against the natural development will always prove an expensive disappointment.

Perhaps in these cases of channel-erosion still more than along a wave-controlled shore, once a fixed structure has been established, it will have to remain functioning, quite probably have to be extended later on and certainly have to be looked after from here to eternity.

It goes without saying that there are any number of intermediate situations, where combinations of the conditions discussed above are in evidence. As this paper deals with general principles only, the possible combinations will not be enlarged upon herein.

The moral of the contents of this paper is that in coastal defence one should do as little as possible, in particular as regards construction of fixed structures and, if one cannot avoid them, design them to do as little harm as possible.

To be somewhat more explicit:

1. If an erosion problem has to be tackled, make very sure in the first place, that it is real, that is to say permanent, erosion and not just a passing eroding phase, which may be expected to be followed by a phase of restoration.

2. Enquire very thoroughly into the first cause of the erosion and see if this can be remedied or mitigated. In cases where human activities are responsible for the erosion, this can often be done. When planning new developments try to design them so as to limit their erosive effects to the utmost. Prevention is better than healing.

3. Study the mechanism of the coast and try to distinguish between the effects of sand displacement perpendicular to the coast, longitudinal drift, and the action of currents.

4. Whenever technically possible and economically feasible, accomplish the remedy artificial supply rather than by fixed structures.

5. If fixed structures must be made, design them very carefully to fulfil just their purpose and to have no additional effects, which may make them do more harmful than beneficial.

In conclusion, it appears that the motto of the coastal engineer should be: Be sure to put off till to-morrow what you do not absolutely have to do today.

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TIDAL COMPUTATIONS IN COASTAL AREAS

J. J. Dronkers[1]

SYNOPSIS

The purpose of this paper is the computation of the tidal motion in shallow coastal waters by means of the so-called harmonic method. This method can be applied by the average engineer. As an example the propagation of the tide in Tampa Bay on the coast of Florida is discussed.

INTRODUCTION

Hydraulic engineering in coastal waters, estuaries, inlets, is confronted with the tidal motion and possibly with the propagation of storm surges. It depends on the extent of the structures to be planned, how deeply the engineer will be interested in the tides.

When he is concerned with local structures, so that there will be no serious interference with the tidal motion as a whole, it will be sufficient to collect observational data on the tide as it exists. The influence of the structure on the local flow pattern can then best be investigated by a model of the local situation.

If, however, a substantial interference with the movement of the tides is contemplated, it will be necessary to investigate thoroughly the mechanism of the tidal motion, in order to predict correctly how the intended interference will work out. For this purpose recourse may be had to computations, to research on a model of the entire estuary or to an electric analogue.

In the Netherlands all three of them are used. As each of the methods have their typical merits and limitations, for some purpose one or the other may be more suitable, in other cases they may complement each other when used in conjunction. No attempt will be made to enter upon a comparative discussion of the three methods.

Note: Discussion open until August 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1977 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 1, March, 1959.

1. Chf. Mathematician of the Rijkswaterstaat, The Hague, The Netherlands.

As long as river sections and more or less clearly defined tidal channels are being dealt with, that is to say as long as the path of the flow is determined by the topography, for the mathematical treatment the flow can be regarded as uni-dimensional.

That means that the flow can be described by a set of equations in the x-direction and disregard transverse flow and Coriolis effects.

In this most simple form the differential equations are:

$$\frac{\partial Q}{\partial x} + b\frac{\partial H}{\partial t} = 0 \text{ , continuity equation} \tag{1}$$

$$\frac{\partial H}{\partial x} + \frac{1}{Ag}\frac{\partial Q}{\partial t} + \frac{|Q|Q}{C^2A^2a} = 0 \text{, dynamic equation} \tag{2}$$

Q = discharge (ebb positive)
H = water level above datum
b = storing width of water surface
A = cross sectional area of streambed
a = depth below water surface
C = coefficient of Chezy
g = gradient of gravity

Further, the appropriate boundary and initial conditions have to be added.

There are several methods to solve these equations:

1. Direct methods (numerical integration by finite differences, iteration process).
2. Characteristic method (integration along characteristics).
3. Harmonic methods (Fourier development).

These various methods are dealt with in a paper by J. J. Dronkers and J. C. Schonfeld, entitled "Tidal Computations in Shallow Water (Proceedings of American Society of Civil Engineers, 1955).

The Harmonic Method

Determination of the Equations

In the paper only the harmonic method is considered. From the practical point of view mostly the consideration of two components is sufficient. Then putting

$$H(x,t) = h_0(x) + h_1(x)\cos(wt + \phi_1(x)) + h_2(x)\cos(2wt + \phi_2(x)) \tag{3}$$

$$Q(x,t) = Q_0 + Q_1(x)\cos(wt + \psi_1(x)) + Q_2(x)\cos(2wt + \psi_2(x)) \tag{4}$$

For instance h_1 diurnal tide of h_1 semi-diurnal;
h_2 semi-diurnal or h_2 quart-diurnal;
$w = 0{,}702.10^{-4}$, $w = 1{,}405.10^{-4}$

This is an approximate method, for many cases sufficiently accurate (in

particular when the observational data are incomplete) and relatively easy to carry out.

The average engineer is capable of doing it.

In regions with a fairly regular tidal motion it is usually not necessary to consider more than two components. It is not very difficult to compute the propagation of the components separately.

However, the computations become considerably more complicated if the mutual interference of the two components has to be taken into account. This interference arises from the fact that the hydraulic resistance is not linear but quadratic. If, however, the upperwater discharge may be neglected and the amplitudes of the successive components (h_1, h_2) decrease sufficiently rapid and moreover the range 2 h_1 is small with respect to the depth of the river, in the first approximation this interference can be studied in a rather simple way.

The quadratic resistance may be written:

$$|Q|Q = \pm [Q_1^2 \cos^2(wt+\psi_1) + Q_2^2 \cos^2(2wt+\psi_2) + 2 Q_1 Q_2 \cos(wt+\psi_1) \cos(2wt+\psi_2)] \tag{5}$$

$$= \pm \left[\tfrac{1}{2} Q_1^2 \{1+\cos(2wt+2\psi_1)\} + \tfrac{1}{2} Q_2^2 \{1+\cos(4wt+2\psi_2)\} + Q_1 Q_2 \{\cos(wt+\psi_2-\psi_1) + \cos(3wt+\psi_1+\psi_2)\}\right]$$

The term Q_0, is disregarded, assuming the upperwater discharge of the river is negligible.

Further, in case the tide h_2 is small with respect to the tide h_1 and therefore Q_2 small to Q_1, one may neglect the term with Q_2^2. Then one also may put that the moments of slack water are mainly determined by the h_1 tide so that the moments for which $Q = 0$ may be approximated by $Q_1 \cos(wt + 1) = 0$. Further one may determine a Fourier expansion for the step function ± 1, which is + 1 if Q 0 and - 1 if Q 0. Then it follows:

$$T(t) = \begin{matrix} +1 \text{ if } Q>0 \\ -1 \text{ if } Q<0 \end{matrix} = \frac{4}{\pi} \cos(wt+\psi_1) - \frac{4}{3\pi} \cos(3wt+3\psi_1) + (-1)^{p+1} \frac{1}{(2p-1)} \left(\frac{4}{\pi}\right) \cos[(2p-1)(wt+\psi_1)] \tag{6}$$

Finally the produce of T(t) has been determined considering only the first two terms and the development of Q^2 between the brackets of (5). From mathematical point of view this method may only be applied if a few harmonics are considered.

After some calculation leaving out of consideration the higher harmonics with 3wt and so on, it is deduced that

$$|Q|Q = \frac{4}{3\pi} Q_1 Q_2 \cos(2\psi_1 - \psi_2) + \frac{8}{3\pi} Q_1^2 \cos(wt + \psi_1) + \underline{\frac{4}{\pi} Q_1 Q_2 \cos(2wt + \psi_2)} - \quad (7)$$

$$\underline{\frac{4}{15\pi} Q_1 Q_2 \cos(2wt + 4\psi_1 - \psi_2)}$$

Then the underlined terms determine the influence of the interference of the h_1 and h_2 tide as a consequence of the quadratic resistance.

In virtue of the fact that the number value of $\frac{4}{15}$ is $\frac{1}{15}$ of that of $\frac{4}{\pi}$ and moreover Q_2 is considerably smaller than Q_1, it is allowed to neglect the last term in (7).

Now the linear equations which determine the propagation of the h_1 and h_2 tide can be derived.

For the presentation of the equations there is some advantage in the introduction of complex quantities. Then it may be stated that

$$h_1(x)\cos(wt + \phi_1(x)) = \tfrac{1}{2} h_1(x) \left\{ e^{i(wt + \phi_1(x))} + e^{-i(wt + \phi_1(x))} \right\}$$

$$= \underline{H}_1(x)\, e^{iwt} + \underline{H}_{-1}(x)\, e^{-iwt} \qquad (i^2 = -1), \quad \underline{H}_1(x) = \tfrac{1}{2} h_1(x) e^{i\phi_1(x)}, \qquad (8)$$

$$\underline{H}_{-1}(x) = \tfrac{1}{2} h_1(x) e^{-i\phi_1(x)}, \quad h_1(x) = 2\,|\underline{H}_1(x)|,$$

and analogous expressions for the rest of the goniometric functions in (3) and (4). The purpose is then to calculate the complex functions $\underline{H}_1(x)$, $\underline{Q}_1(x)$, $\underline{H}_2(x)$ and $\underline{Q}_2(x)$, after which also $\underline{H}_{-k}(x)$ will be known

For the term Q Q may be written:

$$|Q|Q = \frac{16}{3\pi} |\underline{Q}_1(x)|\, \underline{Q}_1(x)\, e^{iwt} + \frac{8}{\pi} |\underline{Q}_1(x)|\, \underline{Q}_2(x)\, e^{2iwt} + \text{c.c.} \qquad (9)$$

(c.c. are the conjugated complex functions).

The equations determining $\underline{H}_1(x)$ and $\underline{Q}_1(x)$, are obtained by substituting $\underline{H}_1(x)\, e^{iwt}$ and $\underline{Q}_1(x)\, e^{iwt}$ in (1) and (2), provided Q Q has been substituted by (9). Then it is found:

$$\frac{d\underline{Q}_1}{dx} + ibw\underline{H}_1 = 0; \quad \frac{d\underline{H}_1}{dx} + \frac{iw}{Ag}\underline{Q}_1 + \frac{16}{3\pi} \frac{|\underline{Q}_1|\,\underline{Q}_1}{C^2 A^2 a} = 0 \qquad (10)$$

and for $\underline{H}_2(x)$, $\underline{Q}_2(x)$:

$$\frac{d\underline{Q}_2}{dx} + 2ibw\underline{H}_2 = 0; \quad \frac{d\underline{H}_2}{dx} + \frac{2iw}{Ag}\underline{Q}_2 + \frac{8}{\pi} \frac{|\underline{Q}_1|\,\underline{Q}_2}{C^2 A^2 a} = 0 \qquad (11)$$

The Solution of the Equations

The equations (10) and (11) are two linear homogeneous equations, provided $\underline{Q}_1$ is known.

Now let a channel be divided into a number of sections, so that for each section the quantities b, A and a may be considered independent on the length x. For the j-section the quantities are designated b_j, A_j and a_j.

Further for $\underline{Q}_1$ the mean value in the j-section is considered; its value is estimated provisionally.

After the elimination of $\underline{H}_1$, the equation for $\underline{Q}_1$ is found:

$$\frac{d^2\underline{Q}_1}{dx^2} + \frac{bw^2}{Ag}\underline{Q}_1 - i\frac{16}{3\pi}\frac{bw}{C^2A^2a}|\underline{Q}_1|\underline{Q}_1 = 0 \qquad (12)$$

It is well known that the solutions have the form:

$$\underline{Q}_1(x) = |\underline{Q}_1|\, e^{ux} \quad , \quad \underline{H}_1(x) = |\underline{H}_1|\, e^{ux} \qquad (u \text{ complex})$$

After substitution in (12) it appears that u has to satisfy the quadratic equation:

$$u^2 + \left(\frac{bw^2}{Ag} - i\frac{16}{3\pi}\frac{bw}{C^2A^2a}|\underline{Q}_1|\right) = 0 \qquad (13)$$

Its roots are called u and -u. Then the general solution for $\underline{H}_1$ and $\underline{Q}_1$ of the section considered is:

$$\underline{Q}_1 = |\underline{Q}_1|_1\, e^{ux} + |\underline{Q}_1|_2\, e^{-ux} \,, \qquad (14)$$

and therefore:

$$\underline{H}_1 = \frac{iu}{bw}|\underline{Q}_1|_1\, e^{ux} - \frac{iu}{bw}|\underline{Q}_1|_2\, e^{-ux} \qquad (15)$$

$|\underline{Q}_1|_1$ and $|\underline{Q}_1|_2$ are integration parameters.

With the aid of these relations it is possible to determine the relation between the values $\underline{Q}_{1a}$ and $\underline{H}_{1a}$ at the beginning of a section and the quantities $\underline{Q}_{1b}$ and $\underline{H}_{1b}$ at the end of this section. Then $\underline{Q}_1$ 1 and $\underline{Q}_1$ 2 are eliminated. Similar relations hold good for each section. Further after the elimination of the values of $\underline{Q}_1$ and $\underline{H}_1$ on the borders of the intermediate sections, $\underline{Q}_1$ and $\underline{H}_1$ at the beginning of the first section can be expressed in the values of $\underline{Q}_1$ and $\underline{H}_1$ at the end of the p-th section.

E.g. Let at the beginning of the first section $\underline{H}_1$ be given and let the channel be closed at the end of the p-th section.

Then $\underline{Q}_1$ can be calculated at the beginning of the first section, as well as $\underline{H}_{1p}$ at the end of the channel. Therefore the whole calculation is based on the

elimination of quantities in a system of linear equations. After that the values of $\underline{H}$ and $\underline{Q}$ for all sections can be computed.

At a dividing point of channels it holds good that the water level $\underline{H}_1$ must be the same for the various channels, while the algebraic sum of the currents $\underline{Q}_i$ at this point must be zero, for the total discharge is equal to the total supply.

After that the estimations $\underline{Q}_1$ for each section must be checked. In case the differences are too great, the calculations must be repeated with better estimated values of $\underline{Q}_1$, e.g. with the former computed $\underline{Q}_1$. The calculations must be repeated until there is sufficient concurrence.

The solutions of the equations (11) for the determinations of $\underline{H}_2$ and $\underline{Q}_2$ take place in an analogous way. As the values for $\underline{Q}_1$ for the different sections are known, and this is therefore also the case with the coefficient of the resistance term in (11), these calculations need not be repeated after the first solution. The calculation of the propagation of the second harmonic is therefore much easier than that of the first one.

Example

As an example the propagation of the tide in Tampa Bay on the Gulf Coast of Florida has been considered.

The bay penetrates inland over a distance of about 35 miles. At approximately 20 miles from the mouth the main channel divides into two branches, the western one 15 miles long and the eastern one 11 miles long. The width of the bay below the dividing point is 7 miles and the width of the branches respectively 5 1/2 and 5 miles. Mean depth of main channel is 25 to 30 feet, branches 13 to 17 feet. See Figure I.

The mouth is partly obstructed by some islands.

The vertical tide at the sea shows considerable variations. In the course of a year nearly pure diurnal tides as well as nearly pure semi-diurnal tides occur and also every conceivable intermediate pattern.

Data for the computation were available from Topographic Chart 1257 U.S.C. and G.S; heights and times of HW and LW are known at St. Petersburg from Tide Tables, East Coast North and South America, 1955. Ranges of mean and spring tides and time lag with respect to Tampa Bay at six stations follow:

Station	Ranges (ft.)		Time
	mean	spring	h.m.
1. Egmond Key	1,3	1,7	-2,25
2. Anna Maria	1,4	1,9	-2,20
3. Bradenton	1,4	1,9	-0,45
4. Tampa Bay (St. Petersburg)	1,4	1,6	0
5. Tampa Hillsboro Bay	1,8	2,4	+0,15
6. Safety Harbor	1,9	2,5	+1,45

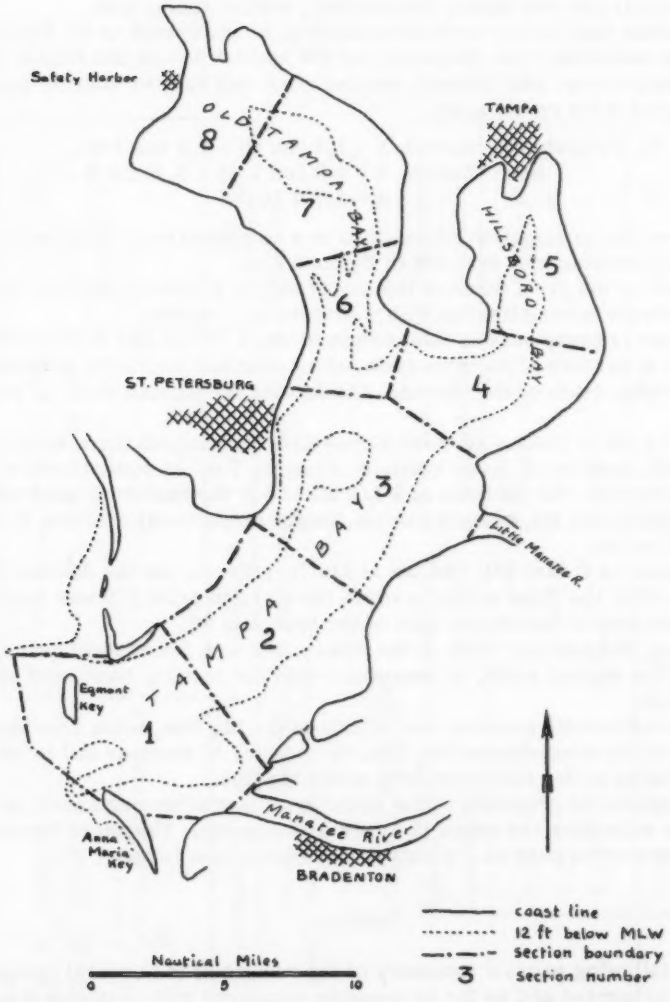

FIGURE I

Tampa Bay, Florida
Network of Sections

In the computations two cases have been dealt with, respectively, mainly semi-diurnal tide and mainly diurnal tide, both at spring tide.

Observed tidal curves were not available, so the curves at St. Petersburg had to be evaluated from the predicted HW and LW levels and times. Making use of time curves with diurnal, semi-diurnal and quarter-diurnal periods, tidal curves were constructed.

E.G. St. Petersburg: diurnal, $h = 1{,}7 \cos wt + 0{,}3 \cos 2wt$,
semi-diurnal, $h = 0{,}8 \cos 2wt + 0{,}1 \cos 4wt$,
$w = 0{,}702\ 10^{-4}$.

From here the propagation of the tides was computed both inland to Tampa and Safety Harbour and seaward to Egmond Key.

In spite of the great width of the bay (7 miles) it proved possible to determine a simple schematization with a number of channels.

In order to carry out any tidal computation, a region has to be schematized into a network of sections each having constant hydraulic parameters, namely width, depth of the channels and further the storing width of water surface.

With a view to reduce as much as possible the computations, large sections were considered; three sections of nearly 7 miles downstream of the point of division, two sections of 5 1/2 miles for the eastern branch and three sections for the western branch, length respectively 6 miles, 6 miles and 3 1/2 miles.

The value of C (see (2)) was put at 110 ft. 1/2/sec. for the downstream sections. For the three sections round the division point 100 was kept and for the sections at the closed ends of the branches 90.

Further, the surface width of the stream bed was put at about half or two-thirds of the storing width, in connection with the existing banks and shallow coast areas.

For a sufficiently accurate calculation of the M_4 tide, being considered together with the semi-diurnal M_2 tide, the number of sections had to be taken twice as large as for the calculation of the M_2 tide.

The transverse gradients of the water level in the bay are small, as the maximum velocities are small (2 ft/sec at the most). Therefore the maximum difference in head as a result of Coriolis forces is about 2".

Results

In the following pages a summary of the results of the several computations are presented and as far as possible compared with available tide data.

At first the case is considered of a semi-diurnal tide (M_2) and quarter-diurnal tide (M_4). A spring tide is assumed.

In the computed amplitudes there are important differences between the results of the calculations and those of the predictions (according to Tide Tables East Coast, 1955).

However, there must be observed these predictions relate to an average tide and do not show the exact tidal conditions this corresponds with, and the relation of diurnal and semi-diurnal tides. A complete comparison is therefore difficult to obtain.

For the above mentioned semi-diurnal tide the situation for the computed tides was considered on 3/4 September 1955 and regarding the diurnal tide

Computed time of propagation from St.Petersburg to:

	Egmond Key	Tampa	Safety Harbor
a M_2 tide	- 175 min.	+ 15 min.	+ 90 min.
b M_4 tide	- 120 "	+ 40 "	+ 125 "
c composed tide, propagation time of HW	- 150 "	+ 25 "	+ 100 "
d predicted time	- 145 min.	+ 15 min.	+ 105 min.

There is a sufficient agreement between the computed and the predicted propagation times.

Computed amplitudes

	Egmond Key (computed)	St.Petersburg (assumed)	Tampa (computed)	Safety Harbor (computed)
a M_2 tide	1,20 ft	0,80 ft	1,00 ft	1,05 ft
b M_4 tide	0,25 "	0,10 "	0,15 "	0,05 "
c composed tide	1,25 "	0,80 "	1,00 "	1,05 "
d predicted increase in % with respect to St.Petersburg	0,95 ft 20%	0,80 ft 0%	1,20 ft 50%	1,25 ft 55%

mentioned below, 19/20 June 1955. The tides considered belong to the strongest ones of their type.

The difference between the calculated compound tides and the predicted tide can be reduced by considering a smaller amplitude of the M_2 tide, by which presumably the average tide is better approximated. This appears from the results of the following tidal calculation, which has been executed in case the amplitude at St. Petersburg for the M_2 and M_4 tides are respectively 0,5 ft. and 0,1 ft.

Computed amplitudes

	Egmond Key (computed) ft.	St. Petersburg (assumed) ft.	Tampa (computed) ft.	Safety Harbor (computed) ft.
a M_2 tide	0,5	0,5	0,6	0,7
b M_4 tide	0,2	0,1	0,2	0,1
c composed tide	0,6	0,5	0,65	0,75
Increase in % with respect to St. Petersburg	20%	0%	30%	50%

The results of the calculated diurnal tide are now presented.

Diurnal tide. Springtide.

Computed time of propagation from St. Petersburg to:

	Egmond Key	Tampa	Safety Harbor
a M_1 tide	- 185 min.	+ 15 min.	+ 70 min.
b M_2 tide	- 175 "	+ 20 "	+ 115 "
c composed tide, propagation time of HW	- 165 "	+ 20 "	+ 110 "

Computed amplitudes

	Egmond Key (computed)	St.Petersburg (assumed)	Tampa (computed)	Safety Harbor (computed)
a M_1 tide	1,75 ft	1,65 ft	1,70 ft	1,75 ft
b M_2 tide	0,70 "	0,35 "	0,40 "	0,35 "
c composed tide	2,10 "	1,75 "	1,85 "	1,90 "

No predictions are available.

CONCLUSIONS

1. Generally the relative decrease or increase along the bay is larger for the M_4 range than for the M_2 ranges; again the latter change more than the M_1 ranges.

2. The propagation time of the M_1 and M_2 tides are practically equal whatever the character of the tide (neap, spring, diurnal or semi-diurnal), it depends mainly on the depth.

3. It is gratifying to note that with a simple method of computation, which can be mastered with relatively little effort by the average hydraulic engineer, with a rather crude schematization of the system and with scant observational data the mechanism of the tidal motion could be explained to a sufficient degree for many practical purposes, provided the data concerning the tides are extended beyond those now available.

Results of the Velocities

The computed mean maximum velocities for ebb and flood in the various sections of the bay are given in the following tables. (See page 24) In the atlas "Tidal Current Charts of Tampa Bay" the hourly "tropic" velocities are mentioned during the tide on many places in the bay. It is not clear with which type of vertical tide they correspond.

Finally, the average maximum values which may be derived from the data of this atlas are presented.

	Section a	Section b	Section c	Section d
ebb	2,5 ft/sec	1,6 ft/sec	0,8 ft/sec	1,8 ft/sec
flood	1,6 "	1,3 "	0,7 "	1,3 "

It appears that there is a fair agreement with the results of the computed maximum velocities for ebb and flood in case of the computed maximum velocities for ebb and flood in case of the tiurnal tide mentioned before.

As has been indicated already, in view of the restricted nature of the available data, the results obtained are valid in broad outline only.

Semi-diurnal tide Springtide

	section a	section b	section c	section d
M_2 tide	1,75 ft/sec	1,30 ft/sec	0,70 ft/sec	1,45 ft/sec
M_4 tide	0,25 "	0,20 "	0,20 "	0,15 "
composed tide:ebb	1,75 "	1,35 "	0,80 "	1,55 "
composed tide:flood	1,90 "	1,35 "	0,70 "	1,40 "

Section a runs from Egmond Key till 6½ miles in the bay,
Section b runs from 6½ miles in the bay till St.Petersburg,
Section c runs from St.Petersburg till 5½ miles in the eastern branche,
Section d runs from St.Petersburg till 6 miles in the western branche.

The computed velocities in the case of the diurnal tide are:

Diurnal tide Springtide

	section a	section b	section c	section d
M_2 tide	1,95 ft/sec	1,30 ft/sec	0,65 ft/sec	1,35 ft/sec
M_4 tide	0,70 "	0,50 "	0,30 "	0,50 "
composed tide:ebb	2,60 "	1,75 "	0,95 "	1,85 "
composed tide:flood	1,70 "	1,05 "	0,60 "	1,00 "

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HYDRAULIC INVESTIGATIONS FOR THE DELTA PROJECT

H. A. Ferguson[1]

This paper describes the field observations, office computations, and model studies made in connection with the hydraulics of the Delta protection and reclamation project now (1958) under construction in the Netherlands. The studies include among other things investigations of tides, waves, currents, salinity, sediment movement, and drift ice.

Previous History and Description of the Project

For many centuries the Dutch have struggled to force from the sea a boundary that would be as favorable to them as possible.

In the northern part of their country they succeeded in crowning this struggle by damming the dangerous Zuiderzee (Fig. 1), which resulted in a better protection of that area as well as in a considerable land reclamation. In the distant future the sea will possibly be driven back even farther northward by means of closing the gaps in the north coastline, which consists of a girdle of islands (Frisian Islands); however, the plans relating thereto have not nearly matured yet.

For the time being the field of operations lies chiefly in the southern part of the coastal area, where the realization of the so-called Delta project has been taken in hand. It provides for the enclosure of the three large and dangerous sea arms situated between Westerschelde and Waterweg, the latter connecting Rotterdam with the sea.

The project represents a set of radical changes to be brought about in a system of sea arms, estuaries and lower rivers, where the tidal movement is considerable and the sandy bottom very unstable. In the area concerned the sea has hitherto had free play, comparatively speaking, and the struggle for the reclamation and preservation of new land has always been extremely heavy.

Note: Discussion open until August 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1978 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 1, March, 1959.

1. Chf. Engr., Rijkswaterstaat, Deltadienst, The Hague, The Netherlands.

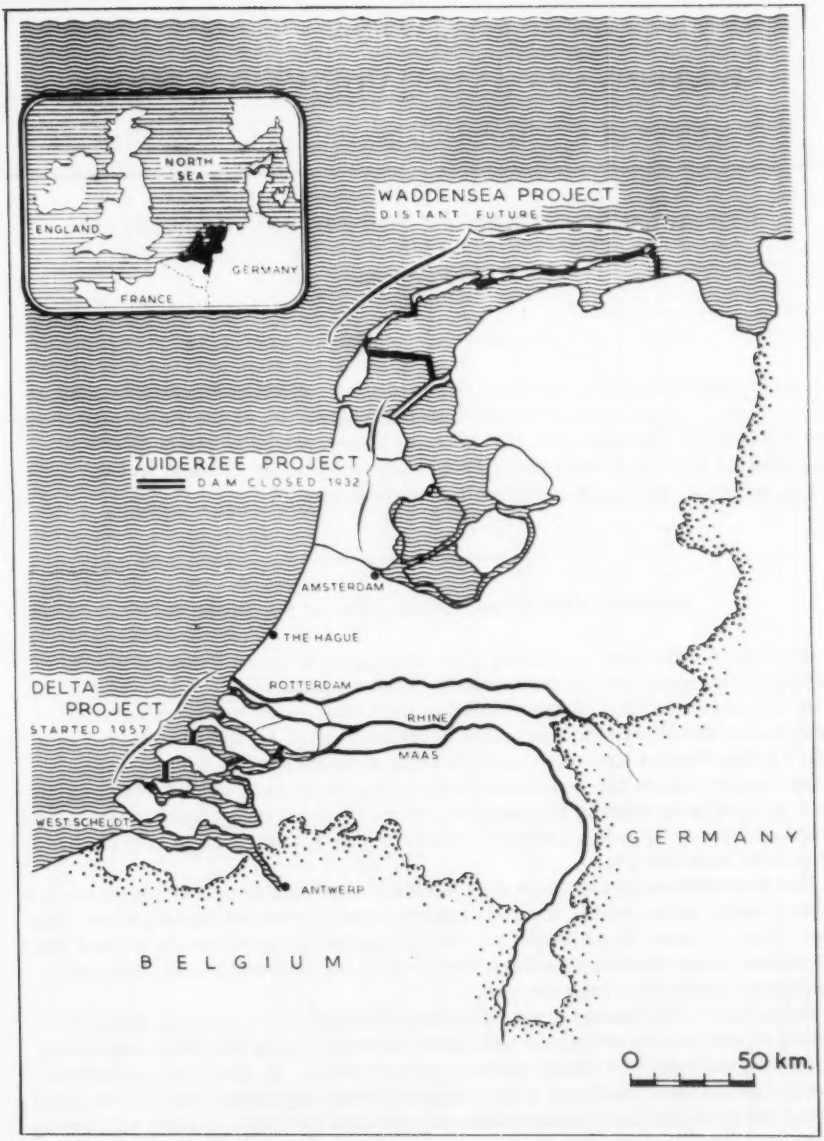


Fig. 1. Projects for Shortening the Coastline of the Netherlands.

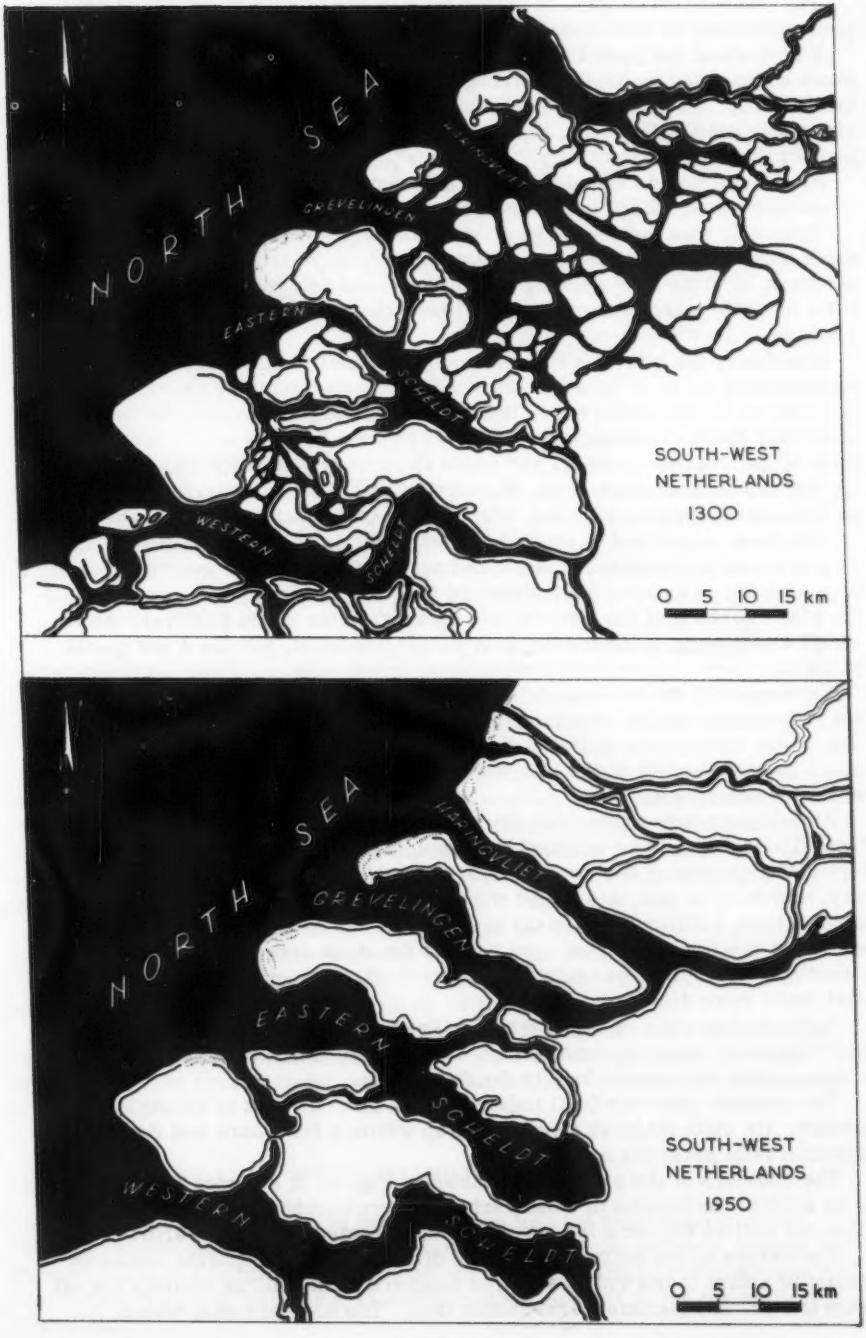


Fig. 2. Historic Changes in the Rhine-Meuse-Scheldt Delta.

The changes which have occurred here in the course of centuries bear plain testimony to that eventful history.

Round about the year 1300, e.g., there lay a rather straggling group of islands surrounded by creeks of irregular shapes, not showing a definite system (Fig. 2). Several of those islands were already inhabited and partly provided with dikes. However, these were far from strong, so that they did not protect the shores effectively.

As the sea could change things at will, several islands disappeared owing to stormsurges, others grew together and new ones came into existence.

Gradually man succeeded in making his influence felt in this process, especially when a system of seawalls began to be constructed. At first these defenses, built as they were by primitive means, did not endure long, but little by little more success was achieved, thanks to improved materials and methods.

Eventually the situation became such as it still is nowadays (Fig. 2), representing more or less a state of balance. Uncommonly heavy losses of land are no longer suffered; on the other hand spectacular accretions do not occur any more. Certain changes do take place, but they are in the main confined to the present channels and come about very gradually. Roughly speaking the two northern sea arms, Haringvliet and Brouwershavense Gat, undergo a slight sanding-up process, whereas the Oosterschelde gets deeper.

The dikes which had to protect the low-lying islands against floods were strengthened and heightened again and again. However, the greater security thus effected was partly neutralized by a settling of the soil, a general sinking of the surface of the islands, and a general rise of the sealevel. As a result many parts that once lay above high-tide level, now lie a few yards below it.

Consequently the inhabitants were as yet insufficiently safe against the sea, especially against storm surges. There was a chance, though a slight one, of the occurrence of floods that could not be resisted by the dikes, in which case the safety of the islands, particularly of the very low-lying parts, would be endangered.

An adequate protection from erosion caused by tidal currents as well as from floods can only be effected by completely preventing the sea from entering the dangerous sea arms, which penetrate deep into the land. The only way, however, to gain this object would be by damming off those broad and deep waters, a difficult and risky work of such an enormous scope that it would not readily be decided upon without the most urgent reasons. Nevertheless the feasibility of realizing a project like that was investigated and initial plans were drawn up.

Such was the state of things when in February 1953 the Netherlands were suddenly hit by an exceptionally high stormsurge, which caused many dike breaches and vast inundations in the Delta area.

The disaster gave the final impetus to the birth of what is called the Delta project. Its main features were drawn up within a few years and its materialization is in progress now.

The contours of the project are shown in Fig. 3. It includes three big dams to be built in the mouths of Oosterschelde, Brouwershavense Gat and Haringvliet, as well as two smaller ones to be constructed farther inwards.

The waters of the Delta area will be divided into two separate basins by means of a dam in the Volkerak. The southern basin will be entirely cut off from the sea and become a freshwater lake. The northern one, which

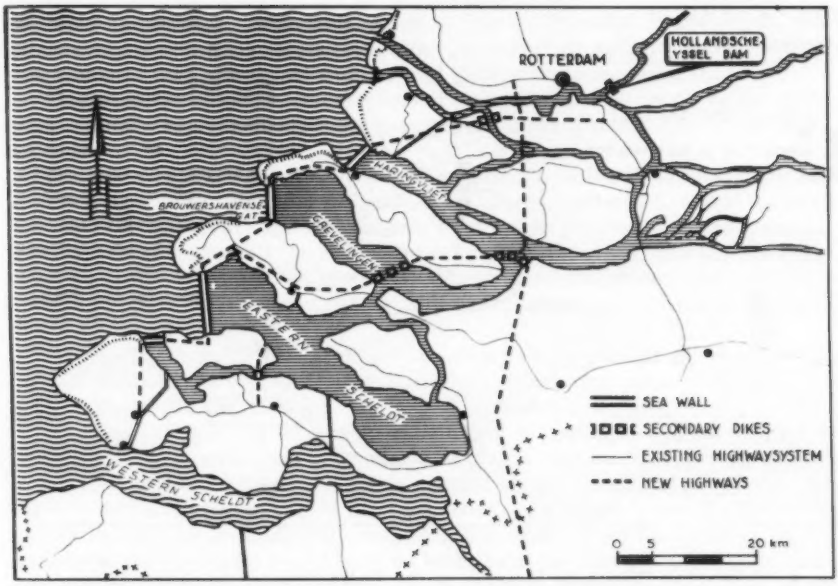


Fig. 3. Layout of the Delta Project.

comprises the mouth of the Rhine and Meuse rivers, will continue to be in communication with the sea, because the Waterweg, connecting Rotterdam with the sea, must remain open to shipping. Consequently tidal waves will still be able to penetrate inland via this mouth, but they will cause high waterlevels in the Waterweg itself only; in the rest of the latter basin their effect will be considerably weakened, as Fig. 4 clearly shows.

It stands to reason that the working out and the execution of the plans confront the engineers concerned with manifold problems, the greater part of which lie in the field of hydraulics. Therefore hydraulic research will play an important part in the realization of the works.

Hereunder follow a number of data on the hydraulic studies and investigations which are being made in connection with the Delta project. The data will not be given exhaustively; only a few outstanding points will be discussed at some length.

Researches into the Tides

Calculations of the tides and investigations into the tidal movement by means of model tests hold a prominent place in the Delta project. Indeed, the project is to a large extent based on them; in its further elaboration and realization, too, they will form an indispensable element in the hydraulic researches.

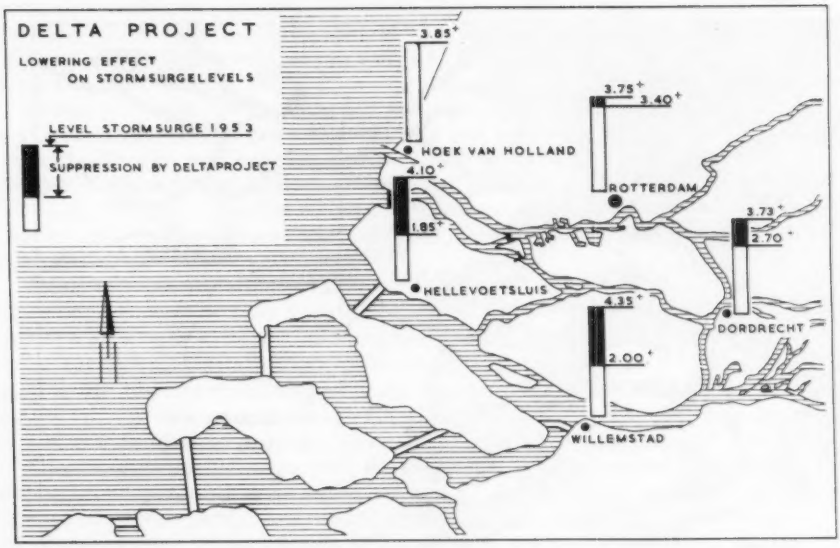


Fig. 4. Lowering Effect, in Meters, of the Delta Project on Storm Surge Levels.

By way of illustration two instances will be given. The first is the study made of the watermovement in the northern basin of the Delta area, the second consists in the research undertaken into the tidal movement in the coastal waters.

Investigations into the Watermovement in the Northern Basin

As has been said before, in the northern basin of the Delta area a tidal movement will remain after the completion of the dams, because the sea will go on entering it via the Waterweg. Besides, the Rhine and the Meuse water will continue to flow off through that basin.

However, the big sluices projected in the Haringvliet will provide a means to control this water movement to a high degree; a sensible arrangement of the sluicing program will ensure a maximum of benefit to, among other things, water management and shipping.

Another means to use the river water as efficiently as possible will be created by the canalization of the northern Rhine branch, a work which is already in progress now.

If necessary a third means of regulating the distribution of the current will consist in the building of a dam in one of the transverse water connections, the Oude Maas ("Old Meuse").

Owing to all these provisions the water movement in the northern basin will differ considerably from what it is now.

In the river branches, whose profiles have adapted themselves to present conditions, the currents will change, so that the profiles may have to be altered artificially.

The drawing up of a plan for the regulation of the water movement on the one hand and the adaptation to it of the system of channels on the other, two factors which naturally affect each other, requires an exhaustive and accurate study of that movement under many different circumstances.

This study is based on calculations of the tides as well as on model tests. Both methods are considered indispensable and are complementary to one another.

The tide calculations afford a better knowledge of the tidal movement and, if carried out with sufficient fullness, they may sometimes produce more exact results. However, the calculations take up very much time and cannot possibly be made for all the cases to be investigated. For this reason they must be confined to a few cases which are to be regarded as highly important from a practical and theoretical point of view; besides, some have to be made for checking purposes.

In a small-scale model far more situations can be investigated in the same period of time than by means of calculations; therefore model tests are especially fit for examining whole series of situations.

At present the latter kind of investigations are mainly carried out in a hydraulic model which has been operated for as long as c. 10 years. However, the tests to be made in this model are rather complicated, so that they take up more time than can really be spared. Moreover it has the drawback that it is not very suitable for examining the effect of profile changes of channels, as the model would have to be altered for this purpose again and again.

Therefore an electric analog is now being developed, with which it is hoped that the comprehensive research program which has been drawn up will be accomplished within a comparatively short time and with sufficient accuracy.

Investigations into the Tidal Currents in the Coastal Area

The tidal currents in the coastal waters of the delta will be subject to considerable changes, when the inlets are closed.

The fan shaped pattern, caused by strong transverse currents, running to and from the estuaries will then have disappeared, so that a lateral current will prevail.

This change will undoubtedly also cause important alterations of the movement of sediments in these waters. It is expected that the large submerged delta of fine sand will gradually disappear, and that the coastline will be flattened out. Such an evolution may be attended with a regression of the slightly protruding capes of the islands, which are without protection against erosion, and built up of sandy materials. Furthermore the alterations in the delta may affect the coast to the north, if the litoral sanddrift from the delta to that part will change.

The process of reshaping and adapting the coastline to the new situation, created by the deltaproject, may take a very long time, perhaps even several hundreds of years. But although slow, this development may cause already in its first stage the above mentioned dangerous effects of erosion. It is

extremely difficult to forecast what will happen and at what rate of time. A comprehensive study of the tidal mechanism, the action of waves and currents, is required to provide the engineers with the knowledge that is necessary to make it possible to detect in time any threat of dangerous erosion and to provide the adequate defensive measures.

Investigation of the hydraulic processes to which coastal waters of the delta will be subject, comprises in the first place a study of the changes in the tidal movements, mentioned above. One of the characteristics of the tidal movement in the coastal waters is the role that is played by the Coriolis force; this is in sharp contrast with the situation in the estuaries where the influence of this force is limited because of the restricted width of the bed, and therefore may be neglected there.

On sea the force of Coriolis is of the same magnitude as the other important factors that command the watermovement in these areas. In this case investigations in a hydraulic model have only sense if the force of Coriolis is introduced. This being so, only a rotating model comes into consideration here.

The application of such a model is under consideration but definite plans for its realisation have not yet been worked out.

Another possibility under study is the application of the abovementioned electrical analog, which probably will offer the possibility of the introduction of the Coriolis force.

For the time being, however, the tidal movement is studied through computations.

The fairly good results, obtained by tidal computations with regard to the present situation, give confidence that in this way a reliable forecast can be made of the future tidal movement when the estuaries are closed.

As only a few different situations have to be considered, the fact that these computations are rather toilsome is not felt as a strong inconvenience; therefore the need of a model is not so strongly felt in this case.

Study of the Sand Movement in the Delta

Forecasts of the changes in the delta, due to alterations of the tidal currents, must be based on a comprehensive study of the combined action of waves and currents that causes the transportation of sediments.

Surveying

The basic information about the sand movements that occur in the delta is to be obtained by soundings. Therefore a methodical and frequent hydrographic survey of the delta is considered of the greatest importance. Consequently a comprehensive survey program for the delta is set up. The most urgent problem to be solved here is the location of the position of the survey-vessels while at work. The surveying has to cover a broad stretch of the coastal waters, and positions have to be fixed at distances up to more than 6 miles from the shore. Exact visual orientation on landmarks, i.e., by means of sextants or rangefinders, becomes very difficult at these distances, and will most of the time be entirely impossible, due to bad view.

Fortunately the modern radio-location systems have made position-fixing independent of visibility. There are several radiosystems, suitable for

surveying purposes. Of these the Decca system of Decca Ltd, London, has been chosen for the surveying of the delta area. This Decca-installation consists of three transmitters, viz. a chief transmitter called master, and two secondary transmitters, called slaves. Their position in the delta is indicated on Fig. 5. Location with the aid of this system is based on the measuring of phase differences between radiowaves, emitted from the three stations, with the receiver placed on board the survey vessel. All spots with same phase differences lie on a hyperbole so that a pair of stations produces a pattern of hyperboles, each of which corresponds with a certain phase difference between the waves emitted from the stations. Thus the ship can fix its position within two nets of these hyperboles, each net based on a master and a slave as focusses.

The hyperbole patterns are shown on Fig. 5.

The accuracy of fixing within this pattern is fairly high, as is shown on Fig. 6 giving the accuracy contours for the Delta survey.

Besides the possibility of a fairly accurate fix at long distances from the shore, the Decca system offers the advantage of a far more efficient surveying than was possible in the orthodox way.

In the first place, Decca offers the possibility of running repeatedly exactly the same section lines without having to mark these on shore.

By making the section lines coincide with hyperboles of the Decca net, the location becomes simplest and most exact. The transmission stations are therefore situated in such a way that the hyperbole pattern offers the best opportunity in this respect.

Another advantage of the system is that it makes it quite easy to mark at every turn the same spots along the section lines. Thus an easy and fairly accurate tabular comparison is made possible between successive soundings of the same area.

A special problem is the determination of the waterlevel from point to point while the surveying is in progress. Considering that the tidal range in

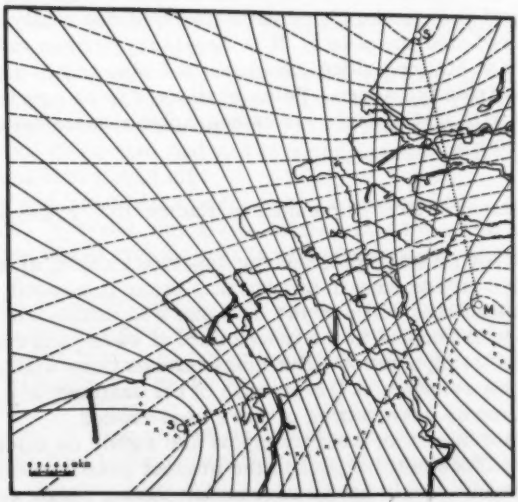


Fig. 5. Hyperbolic Pattern of Decca Survey Chain.

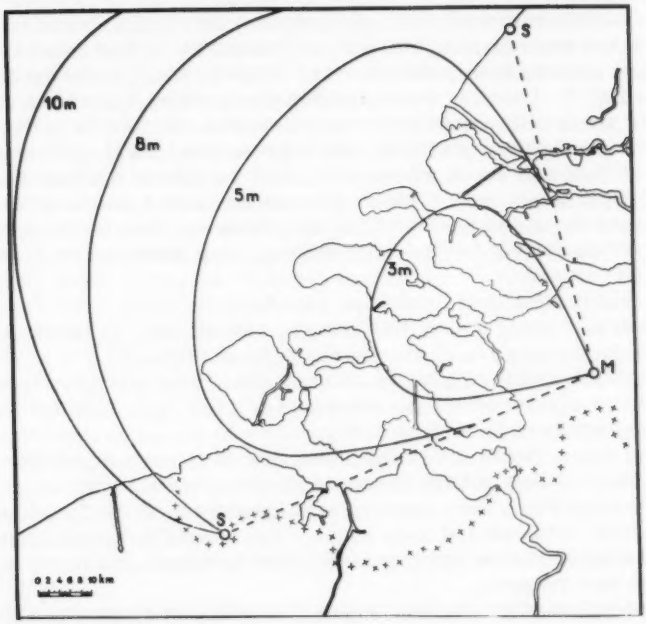


Fig. 6. Decca Survey Chain; Accuracy of Position Fixing.

the coastal area of the delta amounts to more than ten feet, and that there are important phase differences from place to place due to the comparatively slow propagation of the tidal wave in these shallow waters, important mistakes in the estimation of the height of the waterlevel are easily made if the tide is not registered at many points. It is necessary to have registrations of the tide not only at stations along the shore, but also on full sea, at distances many miles from the shore. Poles shall have to be erected for this purpose, equipped with instruments for radiographic transmission of the waterlevel to the shore.

Measurement of Sand Movement by Radioactive Tracers

By the program for hydrographic survey described above, an account will be obtained from the displacements of sand that occur in general in the delta. Without doubt the soundings will provide data that will enable one to draw rough conclusions with respect to the mechanism of wave- and current action that cause this sand transportation. But these informations will not give direct and immediate evidence of the process of the sand movement; they only show where sediments have settled or have been eroded.

For direct measurements of sand transport use should be made of bedloadmeters or suspended loadmeters. But in the present case measurements with such devices would surely not be successful; during rough weather the use of these instruments is not practicable, and it is especially the sandtransport during such periods that is of interest.

Much more success is expected from the use of radioactive tracers, to procure a more detailed view of the sand movement. Application of radioactive tracers in the field has been prepared by modeltests, which tests gave rather promising results.

The tracer that will be used is the so-called greensand (Ionac C 50), which has a specific weight nearly that of the sand in the delta.

The detector used is a scintillation counter, mounted on a sledge on which it can be moved along the seabotton. The greatest difficulty of the tracer method seems to lie in the fact that part of the inserted tracer may temporally be covered by other sand, especially when there are moving sand ripples of some size. In such a case the vertical distribution of the tracer in the soil must be determined also, which will be done by borings; for this purpose a boring device is developed that can procure samples of sand up to a length of 7 feet.

Problems of Bottom Erosion in the Course of the Enclosure Operations

Fig. 7 shows in a diagram how the enclosure of the big estuaries will be executed. The shallow parts and the smaller channels will be bridged first, the main channels afterwards. In this way the operation will proceed gradually, the main current being not much disturbed till at the end, when the remaining deep channels shall be closed by caissons. But of course the current through the remaining gaps will increase considerably in strength as the dam on the shoals makes headway. Moreover, alongside the contracted current a complex of vortextrails and eddies will come into being, as is shown on the diagram. These irregularities in the currentpattern may cause dangerous erosion of the mobile sandbottom, if not special measures would be taken to keep this in check.

In the first place the boundaries of the gap shall be carefully designed, especially the shape of the sill—foundation for the caissons—and the adjoining bottom of the channel.

But this will not be enough; in addition, part of the sandbottom will have to be protected by defensive layers.

As this bottom protection is very expensive, the area to be covered by it should not be exaggerated. On the other hand, no great risks should be taken.

It follows from these considerations that careful studies have to be made of the current and its pattern that has to be expected in and around the gap and of the erosion that it will effect.

To answer these questions model tests are indispensable, but they have their limitations. A model may give a fairly exact imitation of a flowpattern, but the prediction of the erosion that it will effect in nature is much less reliable.

Therefore the relation between current, current pattern and the erosion caused thereby has to be studied in nature. When the connection between current and erosion has been detected in the field, time has come to dray conclusions from currents in a model with regard to erosion, that has to be expected in nature.

Field studies of erosion caused by complex current patterns therefore play an important role in the program of hydraulic investigations for the deltaproject.

An example of these investigations is illustrated in Fig. 8. This represents

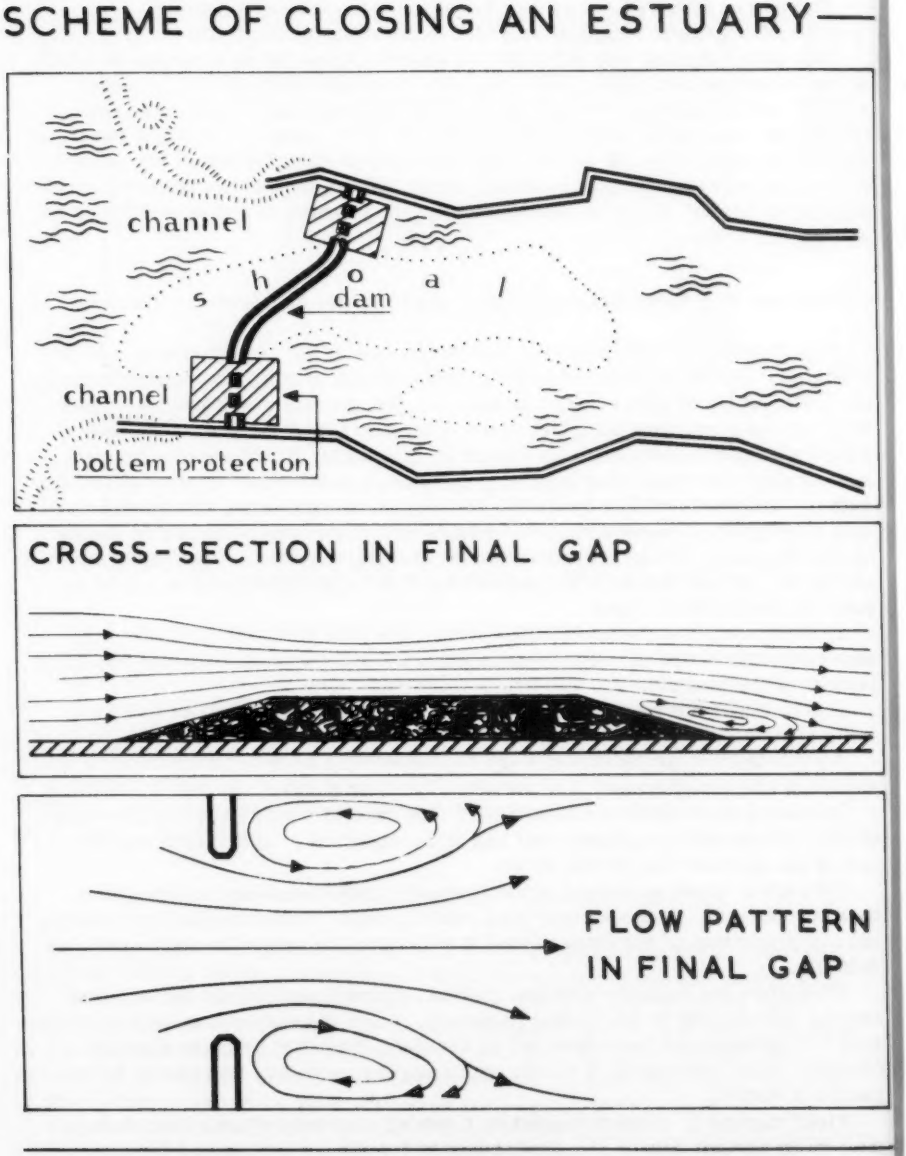
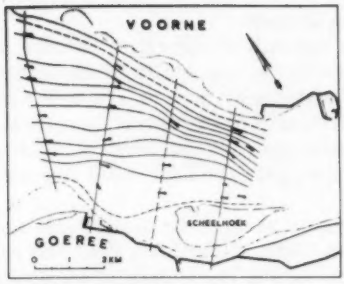
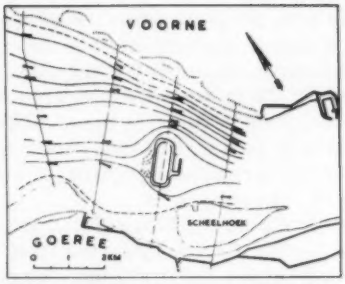


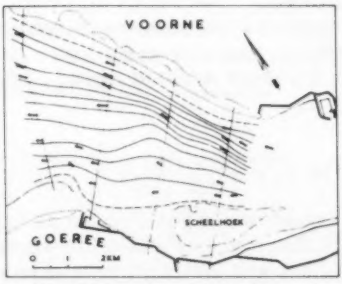
Fig. 7. Scheme of Closing an Estuary.



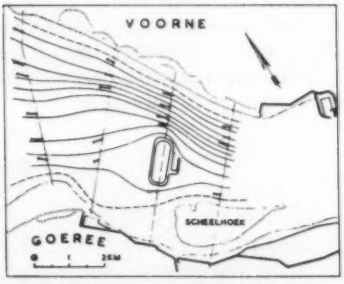
8a. FLOODCURRENT IN HARINGVLIET WITHOUT CONSTRUCTION-PIT.



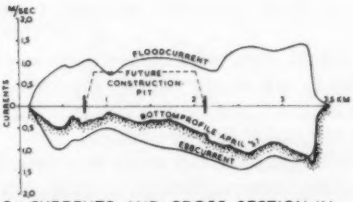
8b. FLOODCURRENT IN HARINGVLIET WITH CONSTRUCTION-PIT.



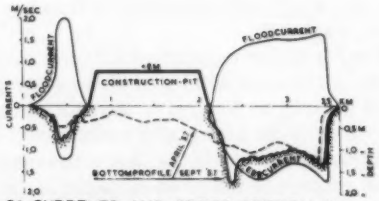
8c. EBBCURRENT IN HARINGVLIET WITHOUT CONSTRUCTION-PIT.



8d. EBBCURRENT IN HARINGVLIET WITH CONSTRUCTION-PIT.



8e. CURRENTS AND CROSS-SECTION IN AXIS OF PIT BEFORE ITS CONSTRUCTION.



8f. CURRENTS AND CROSS-SECTION IN AXIS OF PIT AFTER ITS CONSTRUCTION.

Fig. 8. Flow Pattern in the Haringvliet Near the Construction Pit.

the situation in the Haringvliet, created by building the construction pit for the big sluices, the first phase in the enclosure-operations.

It was evident that such an obstacle, taking about one third of the cross-section of the estuary, would lead to an important change of the flowpattern, and that these changes in their turn will cause considerable erosion in the neighbourhood of the pit.

The flowpatterns of the maximum flood—and ebbcurrent before and after the construction pit was built, are shown in Figs. 8a, b, c and d. In Figs. 8e and f the cross-sections are given in the axis of the pit, before, and some mouths after its construction, with corresponding currentdiagrams.

The current near the northern boundary of the pit is rather turbulent and

from Fig. 8f may follow that there the spot of strongest erosion does not coincide with the place when the current is largest.

For a closer study of these phenomena, not only the average current has to be measured, but also the degree of turbulence, especially of its vertical component. To this purpose, two types of turbulency meters are under development. One of these is based on the principle of pressure measurements and is able to measure not only the horizontal but also the vertical fluctuations of the current, while the other measures only the horizontal fluctuations, with a propeller.

For measuring the flow pattern at the water surface, by floats, use is made of radar. By equipping the floats with radar deflectors, they can easily be detected on the radar screen. By photographing the screen at intervals, the tracks of the floats can be reconstructed in an easy way. The radar applied for this purpose was a 8 mm set from Decca Ltd. London.

This method proved to be easy and fairly exact, and is far to be preferred to the orthodox methods for the location of floats.

Of course the sandmovement has to be studied along with the investigation of the current. Besides frequent soundings, measurements with radioactive tracers are planned, that will procure a more detailed report of the sand displacements in this area.

Wave Research

The enclosure dams will be built in a very advanced position, so as to protect the largest possible area. They will therefore be subject to severe wave attack, as the estuaries lie open to the sea and give easy access to the ocean waves that are generated by the strong western and northern gales that prevail in this region.

The dams will have a main body of fine sand, built up by hydraulic fill and protected only by a rather thin layer of more resistant material (concrete, asphalt).

Even a small damage of this revetment will have serious consequences, as the uncovered sand will be an easy prey to the waves; small gaps will enlarge rapidly as the revetment looses its support and crumbles off. This type of dams is especially endangered by overtopping wave attack.

It goes without saying that a thorough knowledge of the wave movements in the estuaries involved is basic for a wellconsidered design of these works. Therefore a comprehensive program for wave research on behalf of the deltaproject is started.

Study of the Mixing of Salt and Fresh Water

Fig. 9 shows roughly how salt penetration into the northern basin by the Rotterdam Waterway, will be suppressed and how the salt will be flushed back to the sea in cases when the discharge of Rhine and Maas is very small. The sluice in the Haringvliet will be closed then during the whole tide so that all available freshwater, that will not be needed for drinkingwater and agricultural needs, can be discharged through the Waterway. The question that has to be studied here in particular is the benefit of a barrage that shall eventually be erected in the Oude Maas near its confluence with the Waterway. By closing the Oude Maas in times of small riverdischarges all fresh

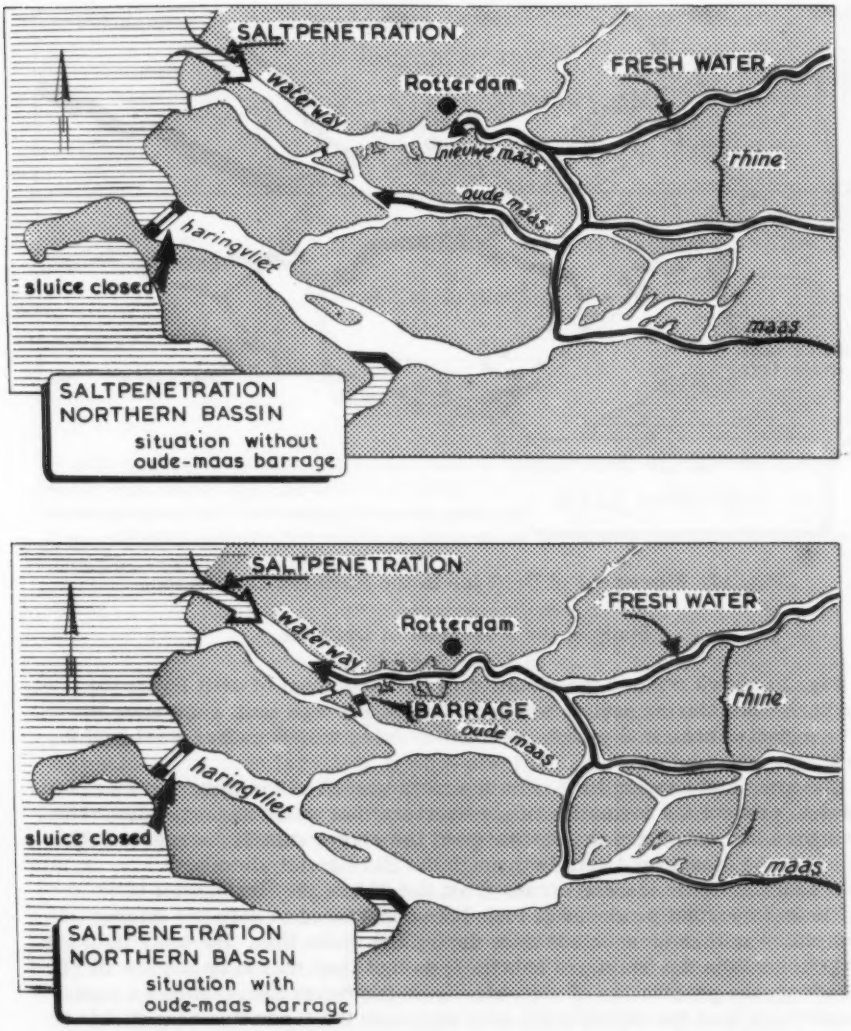


Fig. 9. Suppression of Salt Penetration in the Northern Delta Area.

water available will flow then past Rotterdam. It is to be expected that this concentration upon one branch will have a favourable effect on the suppression of the salt penetration. The question is if this effect will be worth while. It can only be answered after a thorough investigation of the motion of fresh and saltwater in the concerning river branches, a motion that is complicated by the mixing effect of the huge harbour basins alongside the northern branch. This study comprises salinity measurements on the rivers, combined with measurements of the turbulency in the mixing zones, next to which fundamental model tests are foreseen where the principles of the movement of the salt wedge, its turbulent diffusion and the effects of the storing basins will be studied.

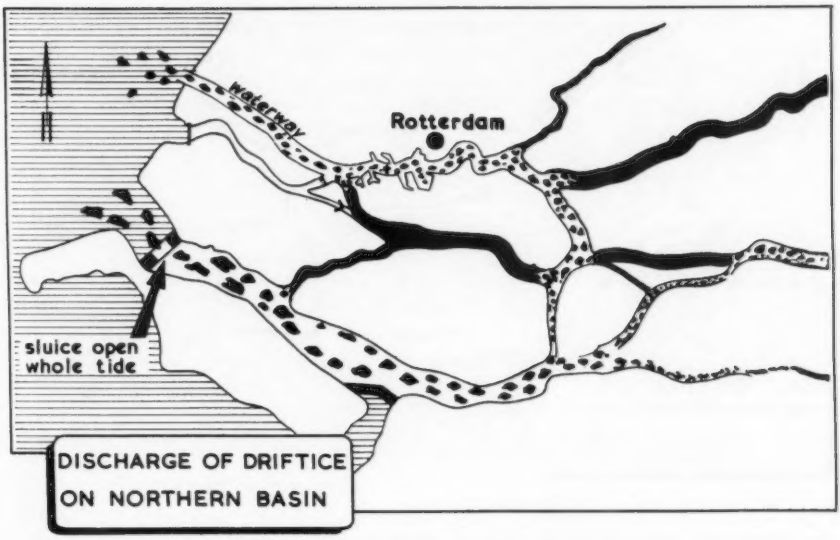


Fig. 10. Movement of Drift Ice in the Northern Delta Area.

Study of the Movement of Drift Ice

Fig. 10 shows roughly the situation of the movement of drift ice in the northern basin during severe winters. On the average such a situation has to be expected once in 5 years, the winters being mostly rather mild in Holland.

The aim is at keeping the ice on the main branch of the Rhine floating, in order to prevent the formation of ice barriers that would endanger the dikes when thaw has set in and the discharge of the rivers rapidly increases. The ice has to be discharged mainly through the Haringvliet sluices.

In addition to the ice that comes from the Rhine, the Haringvliet basin itself is an important producer of ice. It is considered to keep the Haringvliet sluice in these iceperiods open during the whole tide. By doing this, the tidal currents in the basin are enlarged, so that they may keep the ice adrift. Moreover, the penetration of saltwater from the North Sea, that has a higher temperature than the Rhinewater, may suppress the formation of icefields in the basin, especially near the sluices.

However, notwithstanding the open sluice, it cannot be avoided that the tidal currents on Haringvliet and adjoining river branches will be considerably decreased compared with the present situation.

It has to be expected therefore that the discharge of drift ice will meet with much more obstructions when the deltaproject has come into being. It has to be known in time what measures shall be needed to meet this difficulty, so that no dangerous situation will occur in the future. The study of this question includes temperature measurements during ice periods in the present situation, reconnaissances mainly from the air of the movements of drift ice and especially of coloured ice floes, measurements of the relative velocity of the ice floes with respect to the surrounding water, while special investigations will be made into the wind effect on the movement of the drift ice.

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MODEL STUDY OF A HYDRAULIC BREAKWATER OVER A REEF

C. M. Snyder[1]

ABSTRACT

Experimental results of model tests of a combination hydraulic breakwater and submerged breakwater (reef type) are given. Several sizes of reefs were used in combination with a series of hydraulic jet discharges. Experimental relationships among reef size, jet discharge and wave characteristics are presented in graphical form.

INTRODUCTION

Many studies have been made by various investigators on the use of portable breakwater devices.(1,2,3,5) To date the results of the devices tested have not been favorable from a practical point of view. They have been inadequate in calming large ocean waves, have required large moorings to hold them in place, have not been very portable, have been costly, and in cases where power had been used, the power requirements were excessively high.

In the study of one type of hydraulic breakwater at the University of California, a limited number of tests were conducted which used hydraulic jets acting over a horizontal submerged plate. The tests showed results which were better than those of a hydraulic breakwater operating by itself. It was decided therefore to make a more comprehensive study of this system. A submerged reef structure was substituted for the submerged plate. The reefs were made up in various widths and heights, which formed a combination type breakwater, and the results showed that it was possible to get a system that would reduce the power requirement of hydraulic jets from 1/3 to 1/2 of jets alone. The performance of the combination type breakwater showed a considerable improvement over the performance of the reef acting by itself.

Note: Discussion open until August 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1979 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 1, March, 1959.

1. Asst. Research Engr., Inst. of Eng. Research, Univ. of Calif., Berkeley, Calif.

The tests were limited to one value of the ratio of wave length to water depth and to one jet nozzle size. Previous tests[3] showed that it was not practical to use waves that had wave length to water depth ratios greater than six with the existing facilities. Since a large amount of data were available for the ratio of about six (actually L/d = 6.16, where L is wave length and d is water depth), it was selected for the tests described herein.

The use of hydraulic jets may find a practical application in conjunction with existing submerged breakwaters where it is desirable to increase their effectiveness for a short time. They could also be used where existing natural reefs might be utilized as a breakwater.

General Considerations

A hydraulic breakwater is formed by forcing water through a perforated pipe (or a series of nozzles). The resulting jet discharge in the surrounding water induces a current which tends to increase the wave steepness. When the steepness increases to a wave height to wave length ratio of about 1/7, the wave will break and lose energy through turbulence. In addition the jet action creates a great deal of turbulence which dissipates some of the wave energy.

A submerged reef type breakwater is a barrier which has its top at, or below, the still-water level. The breakwater causes some of the wave energy to be reflected out to sea and, in addition, if high enough, causes the wave to break with a dissipation of energy. The forces on such a breakwater are less than those found on the conventional type breakwater.

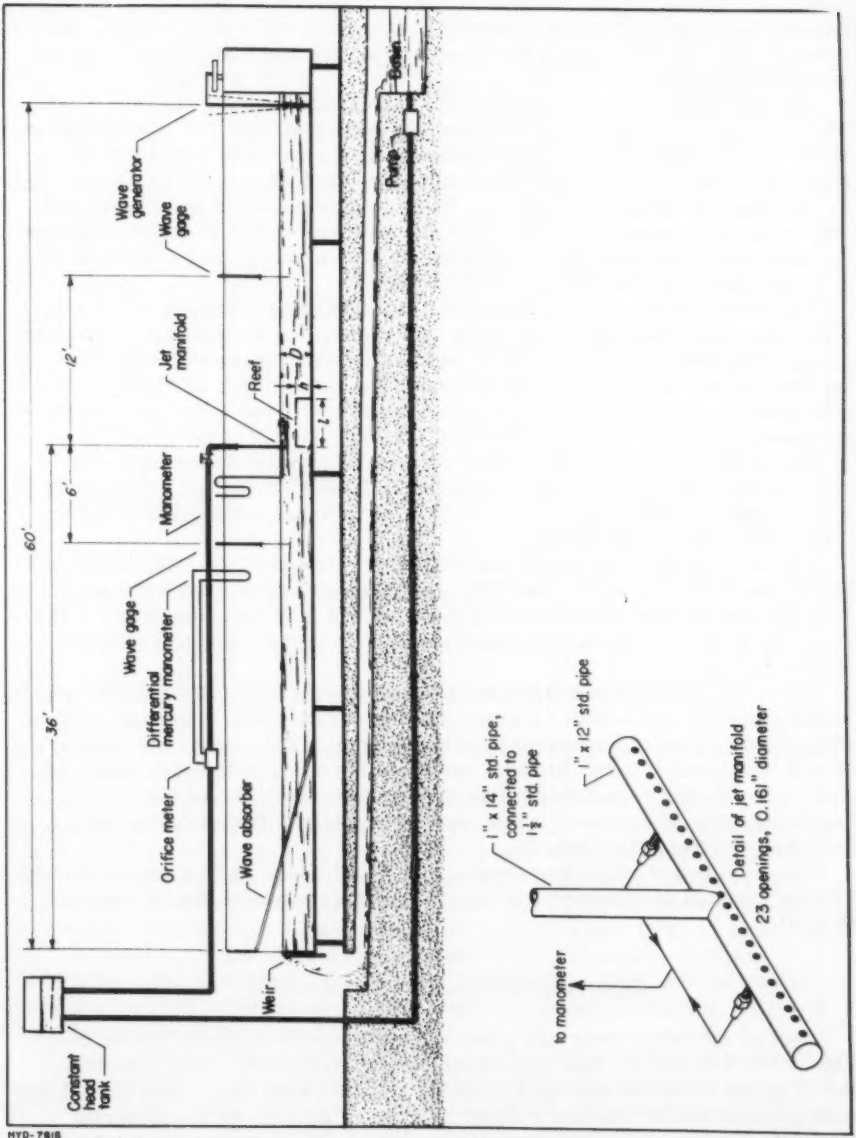
Previous tests had been made using a recirculation pump with its intake seaward of the breakwater. The test described herein utilized a constant head tank so that a better control of the jet discharge could be maintained. The water height was maintained constant in the channel by using a weir at the shoreward end of the channel and by opening a drain valve at the seaward end of the channel. In this manner a nearly constant water level could be maintained.

All previous tests have showed that the use of a hydraulic or pneumatic breakwater was ineffective in attenuating shallow water waves unless relatively large amounts of power were used. The process of reducing wave heights by these methods was found to be very inefficient.

The possible use of a combination type breakwater, where a hydraulic jet is used in conjunction with another type of breakwater, might provide an effective and less expensive way of reducing the wave height. The use of hydraulic jets may also increase the effectiveness of existing natural or man-made breakwaters. A submerged reef-type breakwater was selected because this type of structure is found as a natural breakwater in many places and is a less expensive type to construct in cases where it must be built. Many breakwaters may be submerged at high tide and for this reason can be considered submerged reef-types. If a hydraulic jet manifold could be installed above the reef, or allowed to float on the water surface, it would be possible to increase the effectiveness of the reef as a breakwater.

Laboratory Apparatus

The experiments were conducted in a channel 1 ft. wide by 3 ft. deep by 60 ft. long.[4] The channel had a flapper type wave generator at one end



HYD-7818

FIGURE 1

which was capable of producing waves up to 6 inches high and 20 feet long. At the other end of the channel a wave absorber was installed which consisted of expanded metal baskets packed with aluminum shavings. These baskets were set up to form a sloping beach which had a 1 to 5 slope. The channel was left open behind the beach and a weir was installed to keep the water level constant (Fig. 1).

The jet manifold was made of 1 inch standard pipe in the shape of a tee. The horizontal portion of the manifold had twenty-three 0.161 inch diameter holes equally spaced on 1/2 inch centers. On the opposite side from the holes were two 1/8 inch pipe taps which were used as pressure taps in order to measure the manifold pressure. The manifold pressure was measured with a mercury manometer, one side of which was vented to the atmosphere. An orifice was installed in the atmosphere vent to damp the oscillations that occurred when the jets discharged into waves.

The water supply to the jet manifold was furnished through a two-inch pipe from a constant head tank which was kept full by an existing laboratory pump. The flow to the jet manifold was measured by means of a mercury manometer which was connected to an orifice meter in the two inch line. It was possible to obtain flow discharge rates up to 0.1 cubic feet per second and manifold pressures up to 25 feet of water.

The reefs used as barriers were made up of plywood in box form with steel weights mounted inside the boxes to hold them in place. These reefs were made in widths of 0.67, 1.33, 2.50, and 3.00 feet and heights of 0.30, 0.40, 0.50, 0.60, and 0.70 feet.

For most of the runs the jet manifold was mounted at the stillwater level and at the shore end of the reef (Fig. 1). However, a few runs were made with the jets located at distances of 0.67 feet and 1.33 feet shoreward of the reef, and a few runs were made with the jets located 1.0 inches below the still-water level.

The wave heights were measured by parallel-wire type resistance wave height gages.(6) Each gage was connected in parallel with a 10 ohm resistor which formed part of a wheatstone bridge circuit. The wheatstone bridge circuit consisted of three 100 ohm resistors for three of the legs, and a 90 ohm and a 10 ohm resistor (mentioned above) connected in series for the fourth leg. The wheatstone bridge was connected to a Brush Universal Analyser and Recorder (Fig. 2).

Two wave height gages were used. One was located 12 feet seaward of the jet manifold and the other was located 6 feet shoreward of the jet manifold (Fig. 1).

Experimental Results and Discussion

A set of reference runs was made without the reef in place. These runs were made with the jet manifold located at the still-water level. The water was supplied to the jet manifold from the constant head tank. Data from these runs established the reference curve with which all data were compared. This reference curve is shown on all graphs. In these graphs H_I is the incident and H_T is the transmitted wave height.

The graphs presented show the wave attenuation in per cent as a function of jet discharge in cubic feet per second per foot of breakwater, and they also

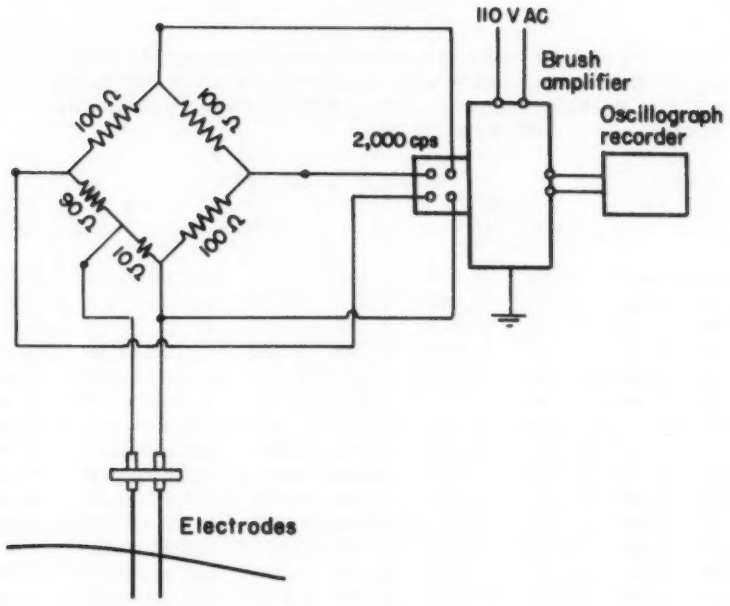


Fig. 2. Wave height measurement with two electrodes

show the wave attenuation in per cent as a function of jet horsepower per foot of breakwater (Figs. 3 to 19).

In Figs. 3 to 7 the data, as presented, show the effect of increasing the reef's width when the reef's height was held constant. The data presented in Figs. 8 to 12 show the effect of increasing the reef's height when the reef's width was held constant. Fig. 13 shows the differences between data of previous tests(1) and the present series of tests. The effect of locating the jets shoreward of the reef are shown in Figs. 14 and 15. Figs. 16 to 19 show the effect of locating the jet below the still-water level. A summary table (Table I) has been prepared from the data presented in Figs. 3 to 12 which shows the relative performance of all breakwater combinations for the wave attenuations of 50%, 60% and 70%.

When the data in Figs. 3 to 7 and Table I were compared it was found that the best combination of hydraulic breakwater and submerged breakwater was the 0.70 high by 3.00 ft. wide reef. The reef did not require any jet discharge to obtain a 50% attenuation, and the 60% and 70% attenuations of the jet discharge and the jet power was less than that of any of the other reefs tested. It was possible to obtain wave attenuation as high as 80% with this combination of hydraulic and submerged breakwater.

The 0.70 ft. high by 0.67 ft. wide reef (Fig. 7) was improved to a greater extent than any of the other reefs by the use of the hydraulic breakwater.

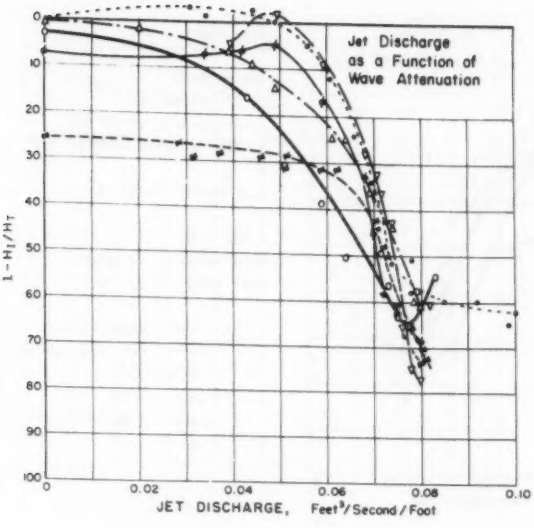
TABLE I

Summary of the Breakwater Results from Figures 3 to 13 Showing Jet Horsepower per Foot of Breakwater and Jet Discharge in Cubic Feet per Second per Foot of Breakwater to obtain Wave Attenuations of 50%, 60% and 70%.

REEF		WAVE ATTENUATION						
Height	Length	50%		60%		70%		Figure
Ft.	Ft.	H.P.	C.F.S.	H.P.	C.F.S.	H.P.	C.F.S.	Numbers
Without reef		0.150	0.073	0.240	0.090	not obtained		3 to 19
0.30	0.67	0.100	0.068	0.130	0.073	not obtained		3 and 8
0.30	1.33	0.125	0.074	0.138	0.076	0.150	0.077	3 and 9
0.30	2.00	0.136	0.076	0.153	0.079	not obtained		3 and 10
0.30	2.50	0.126	0.074	0.148	0.076	not obtained		3 and 11
0.30	3.00	0.115	0.068	0.130	0.065	0.160	0.080	3 and 12
0.40	0.67	0.080	0.066	0.105	0.070	not obtained		4 and 8
0.40	1.33	0.130	0.076	0.153	0.070	not obtained		4 and 9
0.40	2.00	0.155	0.076	not obtained		not obtained		4 and 10
0.40	2.50	0.137	0.074	0.145	0.077	not obtained		4 and 11
0.40	3.00	0.092	0.065	0.124	0.073	0.155	0.078	4 and 12
0.50	0.67	0.084	0.066	0.097	0.069	not obtained		5 and 8
0.50	1.33	0.128	0.074	0.152	0.076	not obtained		5 and 9
0.50	2.00	0.128	0.074	0.152	0.076	not obtained		5 and 10
0.50	2.50	0.133	0.072	0.137	0.074	0.190	0.077	5 and 11
0.50	3.00	0.118	0.073	0.142	0.077	0.160	0.079	5 and 12
0.60	0.67	0.098	0.069	0.128	0.075	0.180	0.083	6 and 8
0.60	1.33	0.116	0.070	0.144	0.078	not obtained		6 and 9
0.60	2.00	0.094	0.064	0.120	0.068	not obtained		6 and 10
0.60	2.50	0.096	0.066	0.120	0.072	0.187	0.076	6 and 11
0.60	3.00	0.035	0.052	0.087	0.064	0.137	0.073	6 and 12
0.70	0.67	0.066	0.061	0.075	0.063	0.086	0.065	7 and 8
0.70	1.33	0.064	0.060	0.107	0.071	0.160	0.080	7 and 9
0.70	2.00	0.050	0.052	0.076	0.056	0.105	0.056	7 and 10
0.70	2.50	0.022	0.044	0.080	0.058	0.126	0.058	7 and 11
0.70	3.00	0.000	0.000	0.006	0.020	0.080	0.065	7 and 12

When the results of the use of the 0.70 ft. high by 0.67 ft. wide reef were compared with the results of the use of the 0.70 ft. high by 3.00 ft. wide reef, it was found that greater jet discharge and greater jet power were necessary to obtain the 50% and 60% wave attenuation for the 0.70 ft. by 0.67 ft. reef; for 70% wave attenuation it was found that the jet discharge and jet power were almost identical for both reefs. When reefs were used in conjunction with a hydraulic breakwater, (Figs. 3-7) it was found that for a given jet discharge increase, the use of shorter width reefs resulted in a greater increase of wave attenuation than did the use of longer width reefs.

The total wave attenuation obtained by the combination of a hydraulic breakwater and a submerged reef type breakwater was found to increase as the reef's height was increased for the same jet discharge and power setting (Figs. 8-12). In all cases the total wave attenuation obtained by the use of the combination breakwater system was greater for the higher reefs than it was for the lower reefs of the same width. As shown in Figs. 6, 7, 8, and 12,



Water depth: 1 foot

Jets at SWL
23 jets per foot
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

Reef height: 0.30 foot

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

without reef
0.67-foot width reef
1.33 " " "
2.00 " " "
2.50 " " "
3.00 " " "

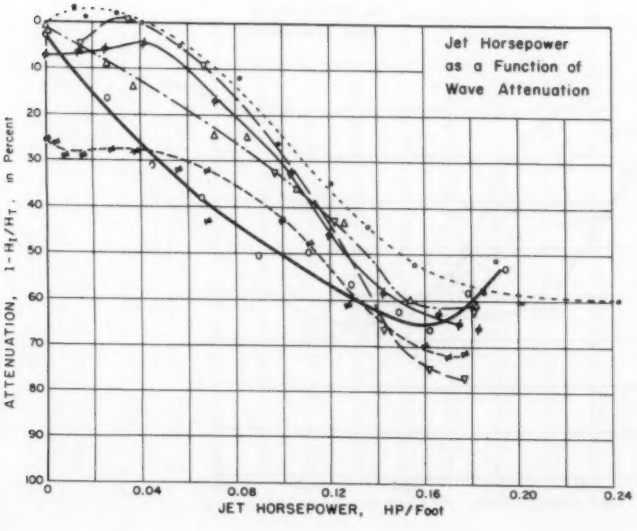
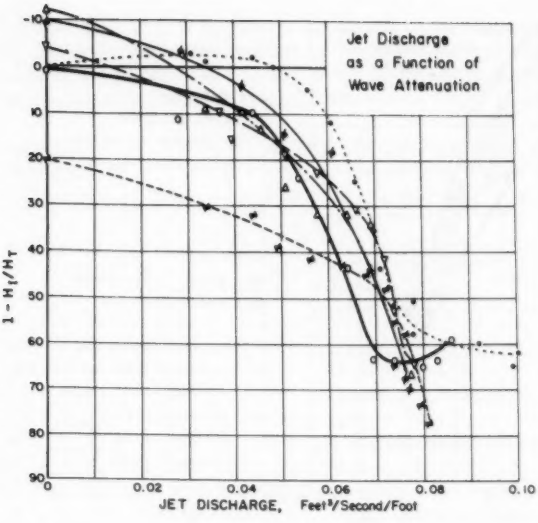


FIGURE 3



Water depth: 1 foot

Jets at SWL
23 jets per foot
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

Reef height: 0.50 foot

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

without reef
0.67-foot width reef
1.33 " " "
2.00 " " "
2.50 " " "
3.00 " " "

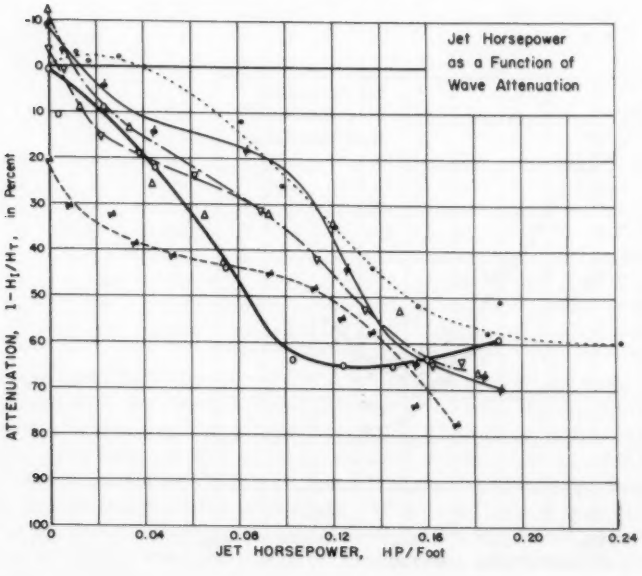


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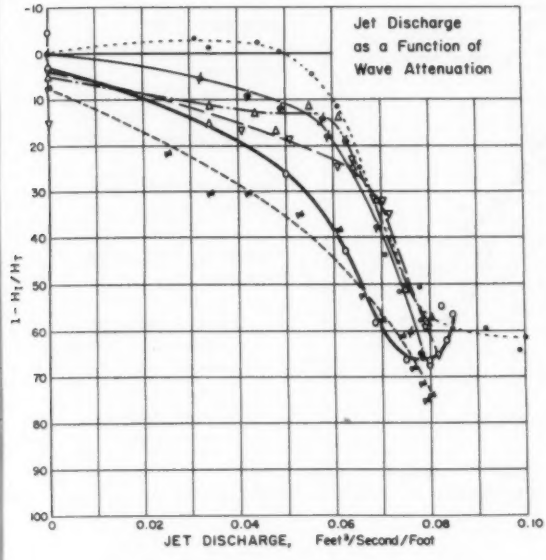
FIGURE 4



Water depth: 1 foot

Jets at SWL
23 jets per foot
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

Reef height: 0.40 foot

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

without reef
0.67 - foot width reef
1.33 " "
2.00 " "
2.50 " "
3.00 " "

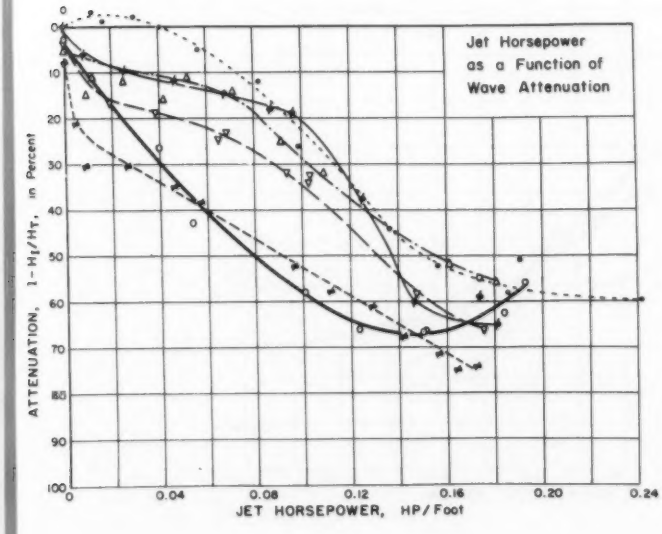
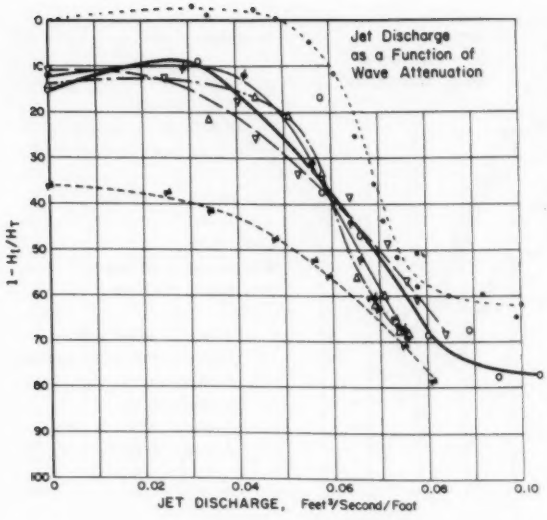


FIGURE 5


Jet Discharge
as a Function of
Wave Attenuation
1 - H1/HT
JET DISCHARGE, Feet²/Second/Foot
0
0.02
0.04
0.06
0.08
0.10
10
20
30
40
50
60
70
80
90
100
Water depth: 1 foot
Jets at SWL
23 jets per foot
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²
Reef height: 0.60 foot
L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot
without reef
0.67-foot width reef
1.33 " " "
2.00 " " "
2.50 " " "
3.00 " " "

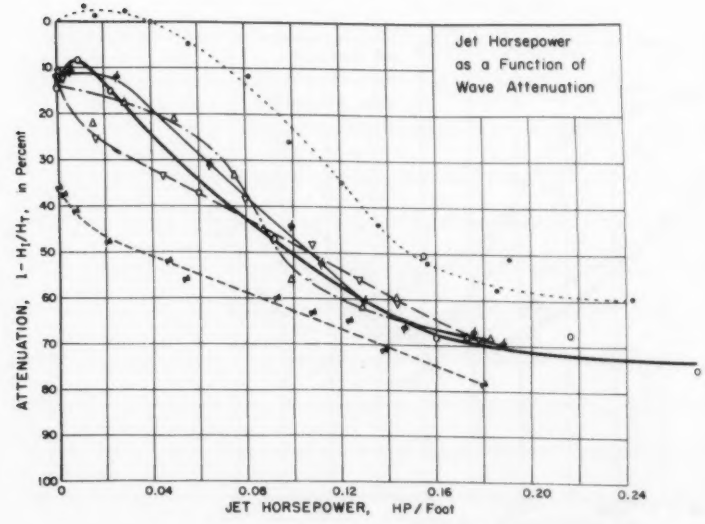

Jet Horsepower
as a Function of
Wave Attenuation
ATTENUATION, 1 - H1/HT, in Percent
JET HORSEPOWER, HP/Foot
0
0.04
0.08
0.12
0.16
0.20
0.24
10
20
30
40
50
60
70
80
90
100

FIGURE 6

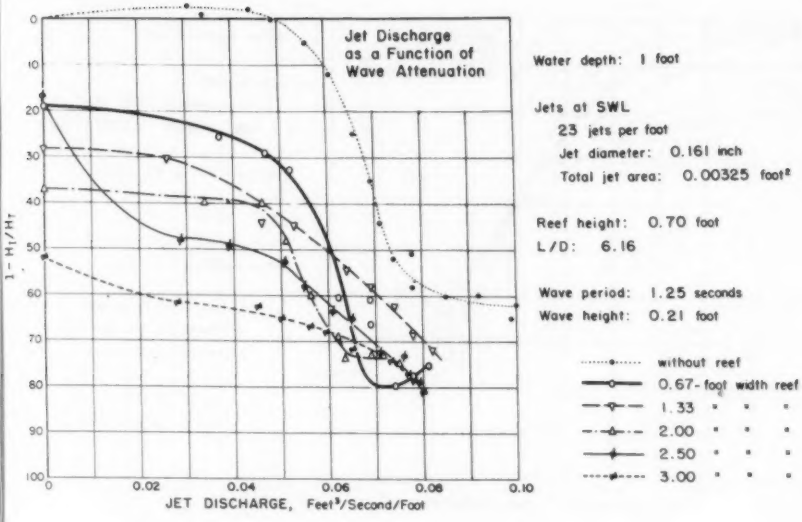


FIGURE 6

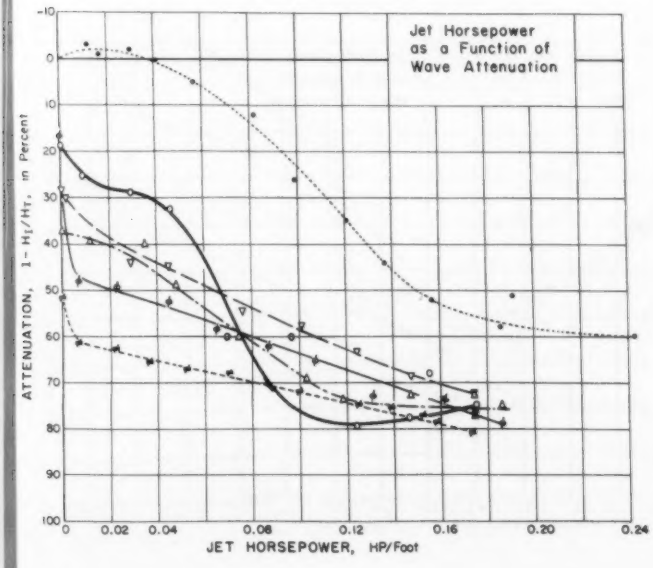
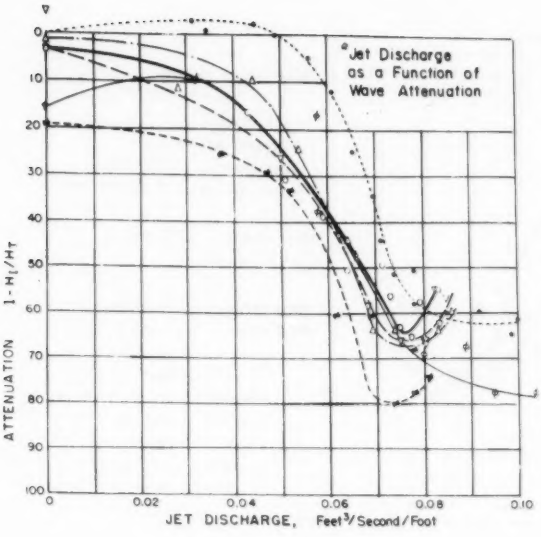


FIGURE 7



Water depth: 1 foot

Jets at SWL
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot

Reef width: 0.67 foot

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

- without reef
- 0.30-foot high reef
- 0.40 " " "
- 0.50 " " "
- 0.60 " " "
- 0.70 " " "

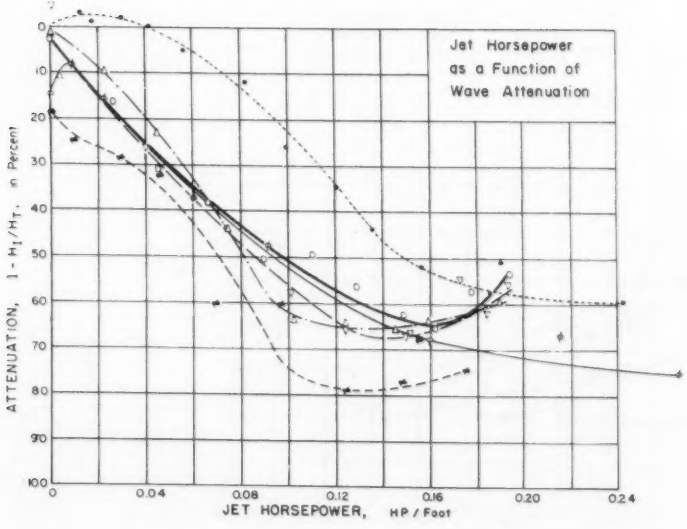
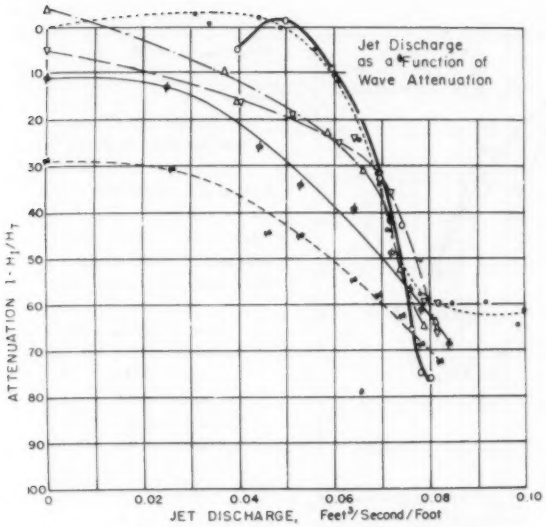


FIGURE 8



Water depth: 1 foot

Jets at SWL
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot

Reef width: 1.33 feet

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

without reef
0.30-foot high reef
0.40 " " "
0.50 " " "
0.60 " " "
0.70 " " "

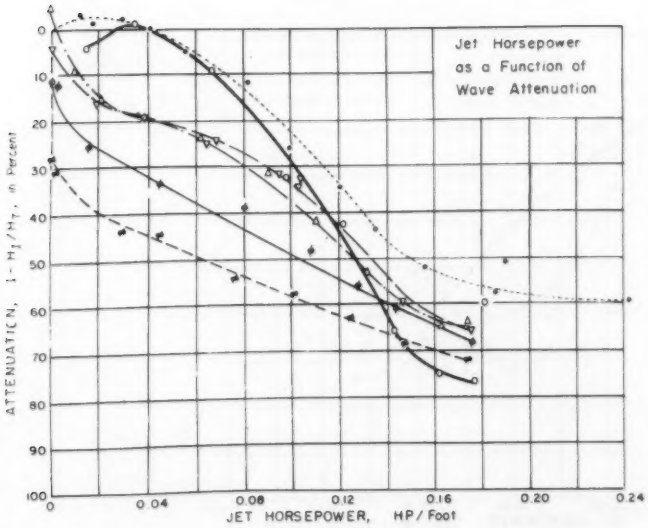
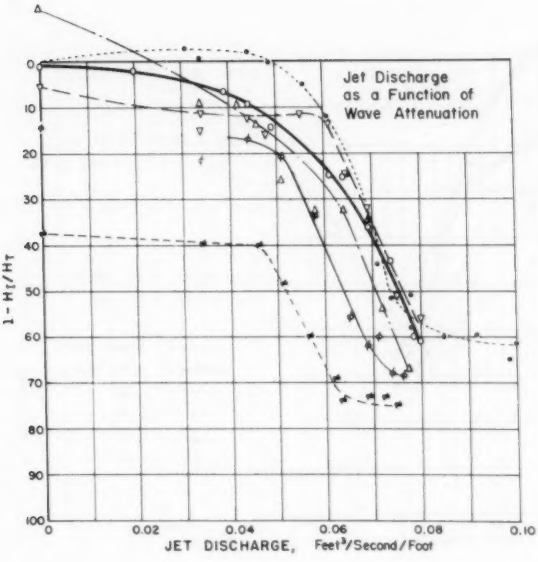


FIGURE 9



Water depth: 1 foot

Jets at SWL
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot

Reef width: 2.00 feet

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

without reef
0.30 - foot high reef
0.40 " " "
0.50 " " "
0.60 " " "
0.70 " " "

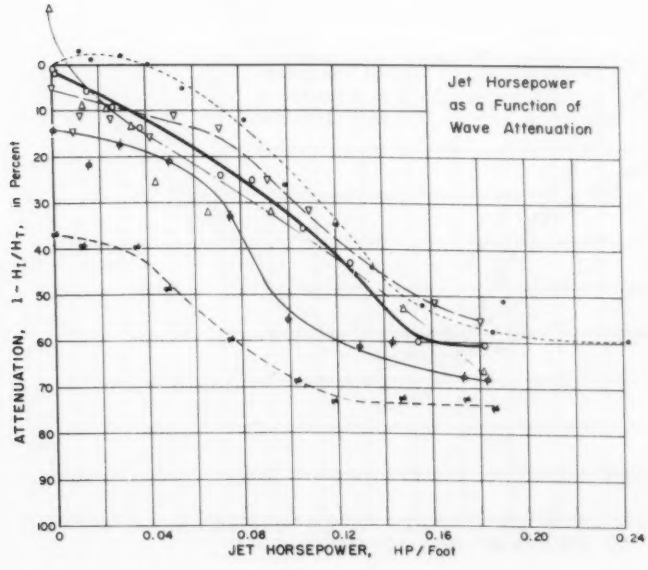


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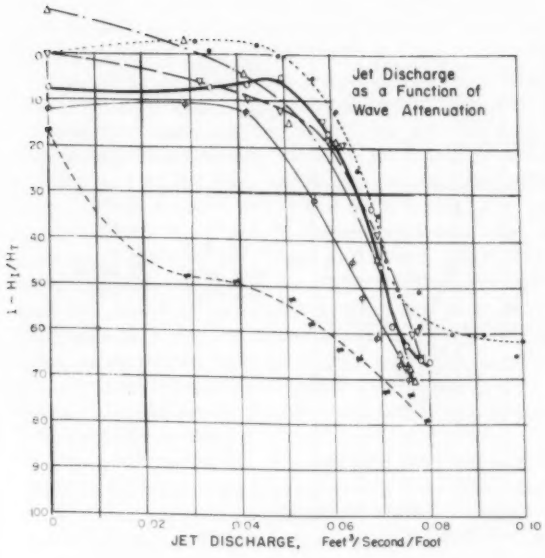
FIGURE 10



Water depth: 1 foot

Jets at SWL
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot

Reef width: 2.50 feet

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

without reef
0.30-foot high reef
0.40 " " "
0.50 " " "
0.60 " " "
0.70 " " "

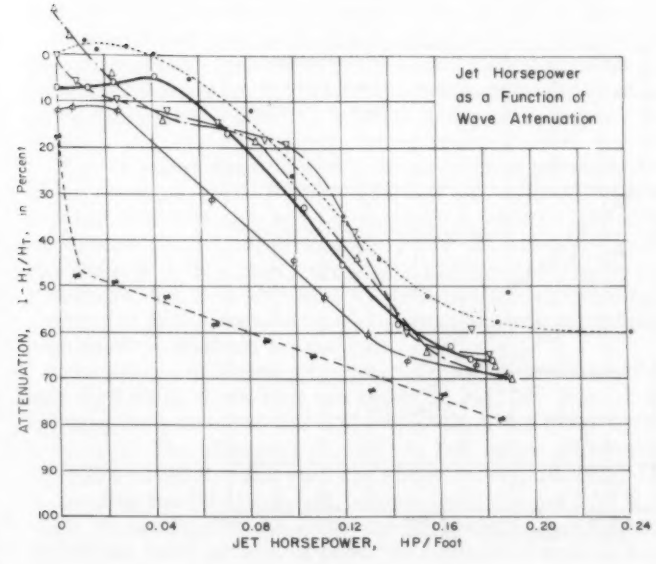
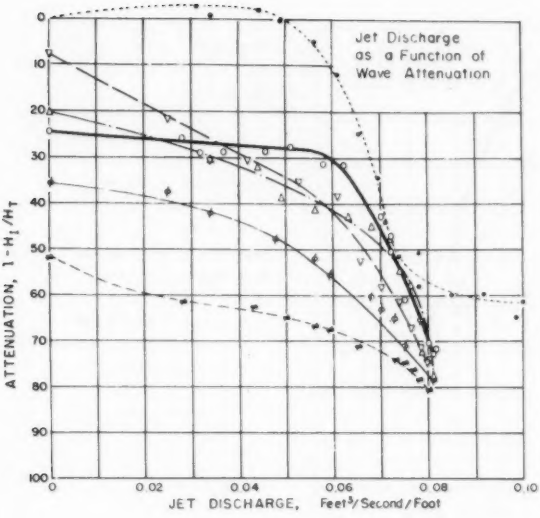


FIGURE 11



Water depth: 1 foot

Jets at SWL
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot

Reef width: 3.00 feet

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

without reef
0.30-foot high reef
0.40 " " "
0.50 " " "
0.60 " " "
0.70 " " "

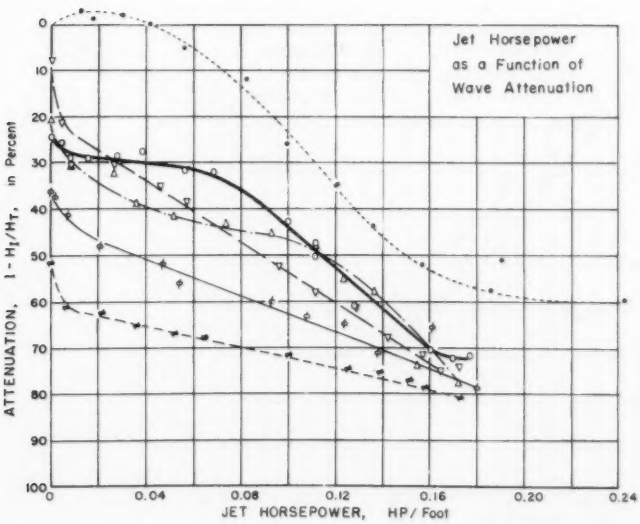


FIGURE 12

the 0.70 ft. high by 3.00 ft. wide reef, the 0.60 ft. high by 3.00 ft. wide reef and the 0.70 ft. high by 0.67 ft. wide reef were the most effective combinations to use with the hydraulic jets.

Fig. 13 is used to show the comparison of data which were obtained from previous test[1] and the present series of tests.

The first series of tests with the jet manifold was made using a recirculation pump,[1] which supplied the jet manifold with water, and the intake of the pump was located seaward of the jet manifold. In the second series of tests (reported in Figs. 3 to 12) the jet manifold was supplied with water from a constant head tank and the water level was kept constant in the channel by the use of the overflow weir and drain valve (Fig. 1). Fig. 13 shows that the recirculation pump system required less jet discharge and less jet power than the constant head tank system for their respective reference curves. The data obtained using the recirculation pump system also showed that less jet discharge and less jet power were required for the submerged horizontal flat plate hydraulic breakwater combination than were required for the results obtained using the constant head tank system with the submerged reef-hydraulic breakwater combination.

The recirculation pump system with its intake located seaward of the breakwater creates a current over a greater distance and in this way may account for the better efficiency shown. The constant head tank system, which discharged part of the water shoreward of the hydraulic breakwater, tended to create more bottom currents in the shoreward direction which, in turn, may have hindered the effectiveness of the breakwater.

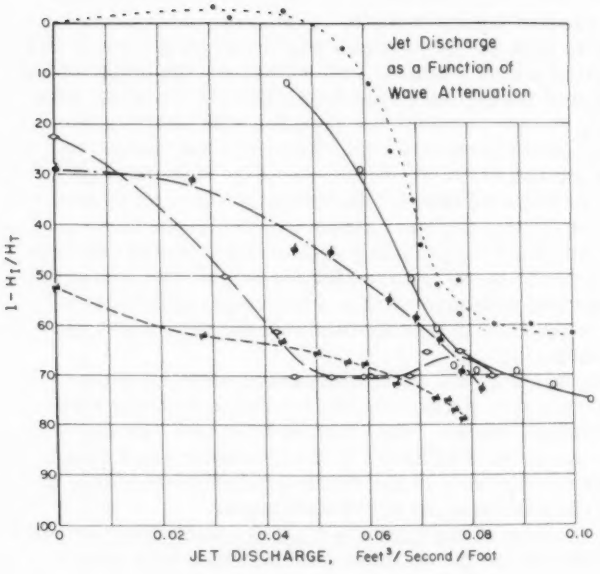
It is apparent that the location of the intake to a pump could affect the performance of the breakwater. Since the constant head tank and weir system were used throughout the tests reported herein, the results were not as good as they might have been if the water had been removed from the channel elsewhere.

Fig. 14 shows the effect of locating the jets shoreward of the 0.50 ft. high by 0.67 ft. wide reef. It was noted that the further back the jets were located the less effective they were in attenuating waves. The effect of locating the jet shoreward of the 0.70 ft. high by 0.67 ft. wide reef is shown in Fig. 15. The results in this case were similar to those shown in Fig. 14. In general, it appears that it is advisable for best performance not to locate the jets shoreward of the reef. However, very few tests were made to determine the best possible position for the jets.

The effects of locating the jets below still-water level for the 0.70 ft. high by 0.67 ft. width reef are shown in Fig. 16. Fig. 17 shows the effects of locating the jets below the still-water level for the 0.70 ft. high by 2.50 ft. wide reef. The effects of placing the jets below still-water level for the 0.50 ft. high by 3.00 ft. wide reef are shown in Fig. 18. Fig. 19 shows the effects of locating the jets below still-water level for the 0.70 ft. high by 3.00 wide reef. These figures show that the still-water level location of the jets required the least amount of power to attenuate waves to a given percentage of their original height. This was also found to be true in experiments reported in Reference 1 for jets acting by themselves.

Prototype Requirements

Prototype values of jet discharge and power requirements were computed using Froude's Modulus. Two sets of tables are presented which show the



Water depth: 1 foot
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²
L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

- Head tank & weir, without reef
- Head tank & weir, 0.70 x 1.33 foot reef
- Head tank & weir, 0.70 x 3.00 foot reef
- Circulating pump system, without reef
- Circulating pump system, with 1.5-foot plate submerged 0.23 foot, jet submerged 0.15 foot

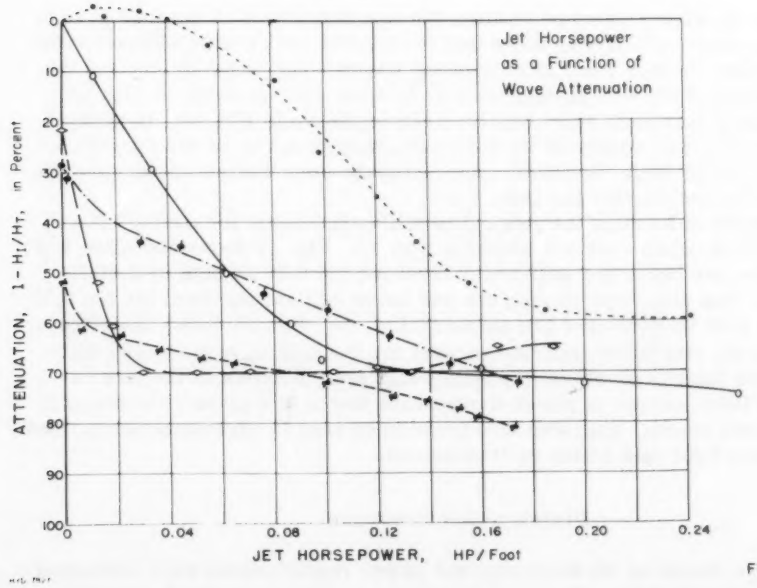
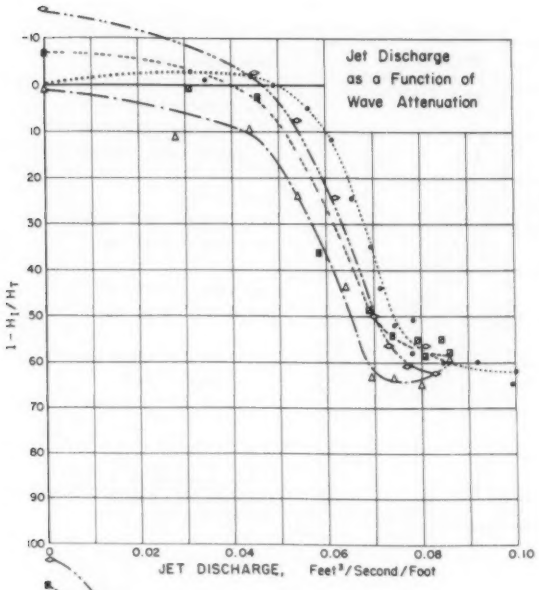


FIGURE 13



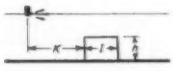
Water depth: 1 foot

Jets at SWL
Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

Reef width: 0.67 foot
Reef height: 0.50 foot

without reef
K = 0
K = 8 inches (0.67')
K = 16 inches (1.33')



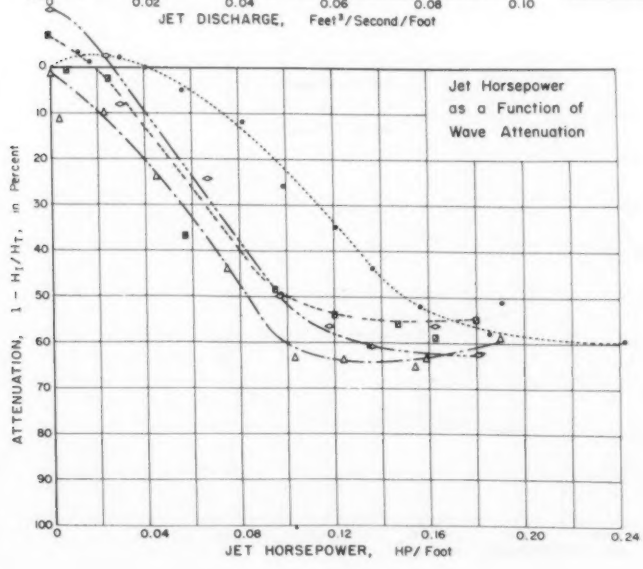
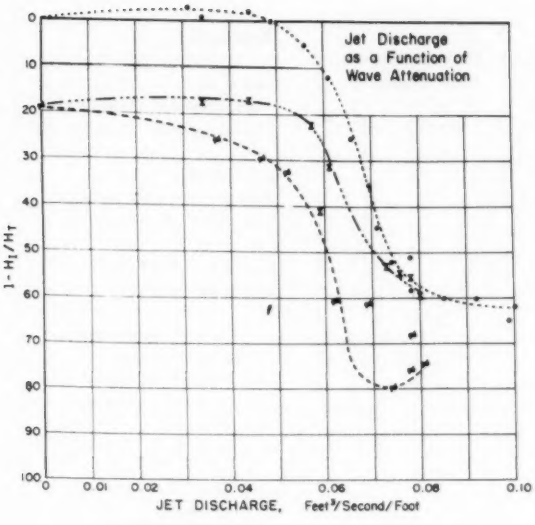


FIGURE 14



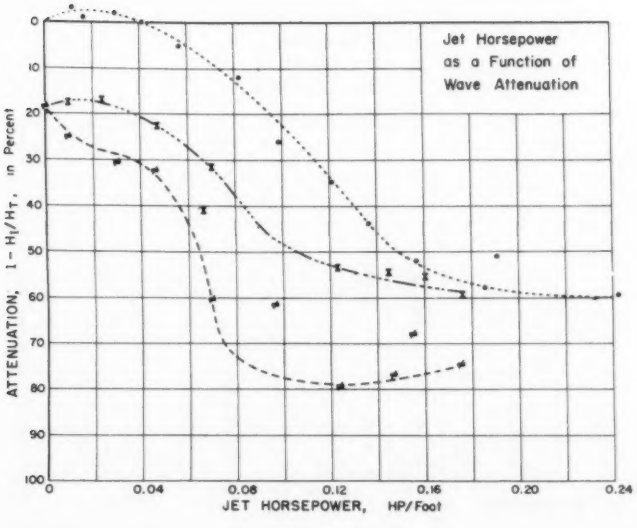


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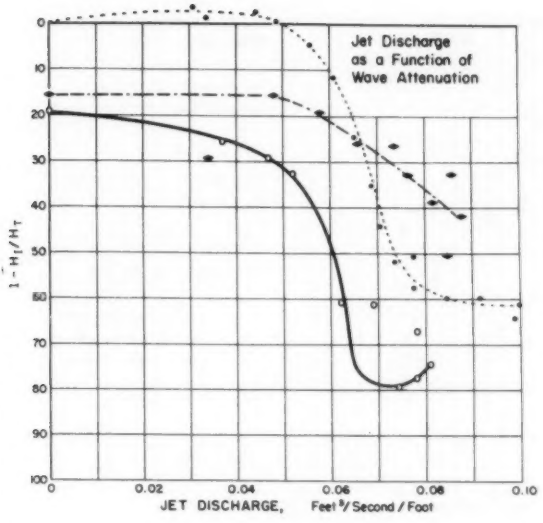
FIGURE 15



Water depth: 1 foot

Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

Reef width: 0.67 foot
Reef height: 0.70 foot

----•---- without reef
——o—— jets at SWL
---•--- jets 1" below SWL

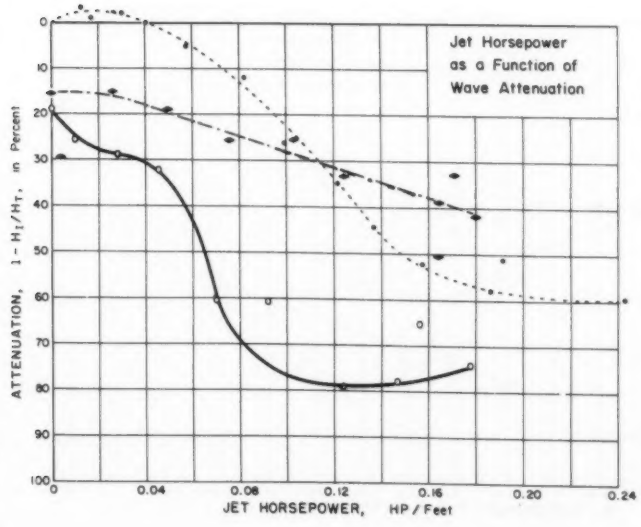
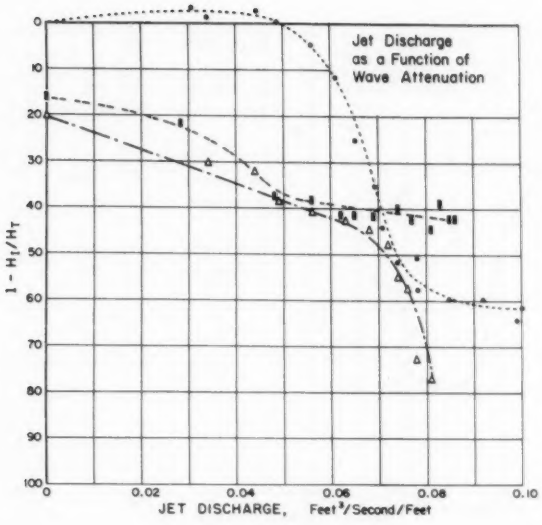


FIGURE 16



Water depth: 1 foot

Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

Reef width: 3.00 feet
Reef height: 0.50 foot

····•···· without reef
—·—o—·— jets at SWL
— — ▮ — — jets 1" below SWL

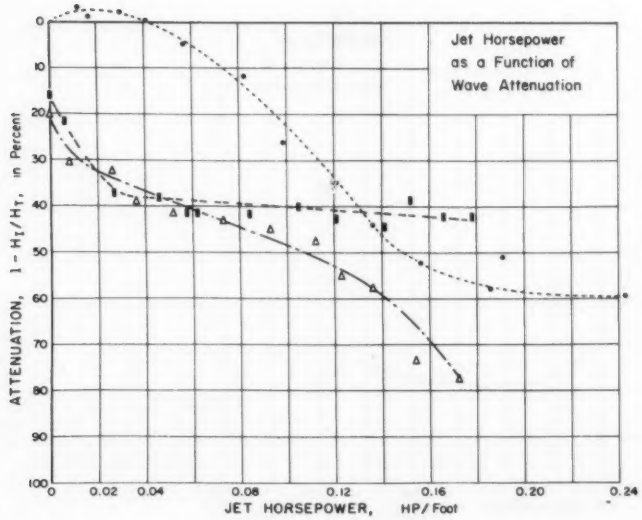
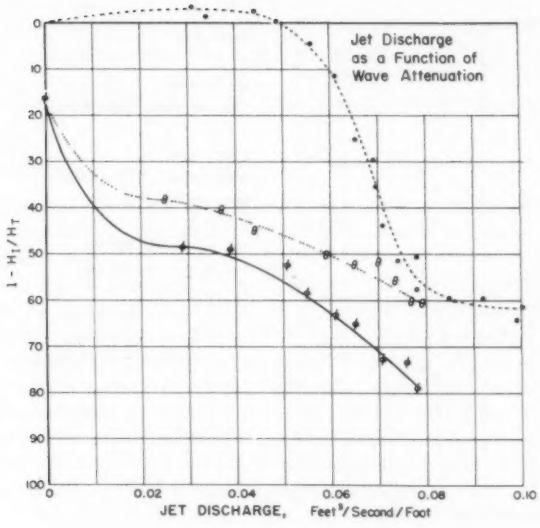


FIGURE 17



Water depth: 1 foot

Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

Reef width: 2.50 feet
Reef height: 0.70 foot

without reef
jets at SWL
jets 1" below SWL

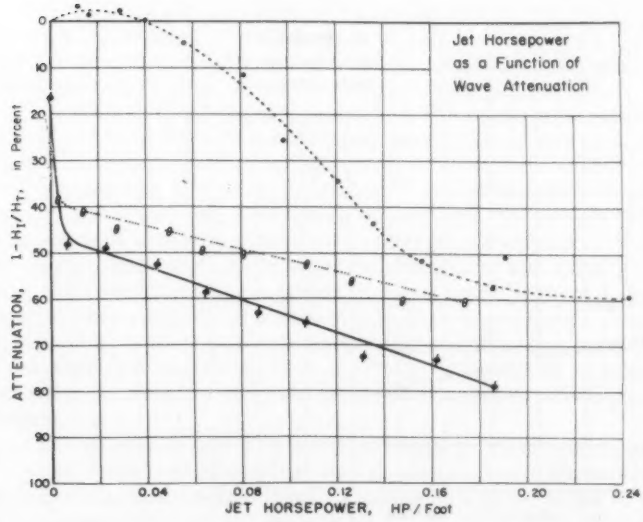
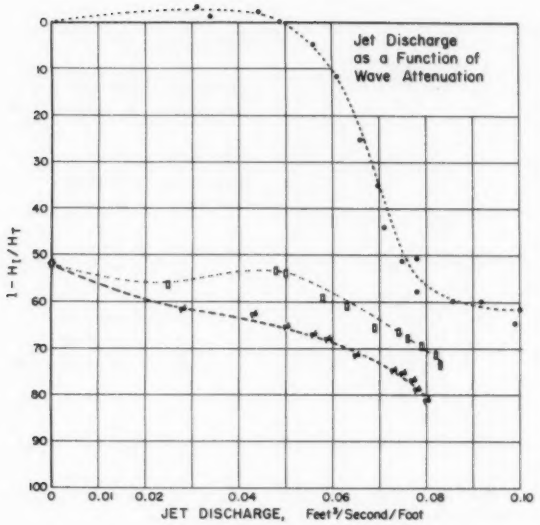


FIGURE 18



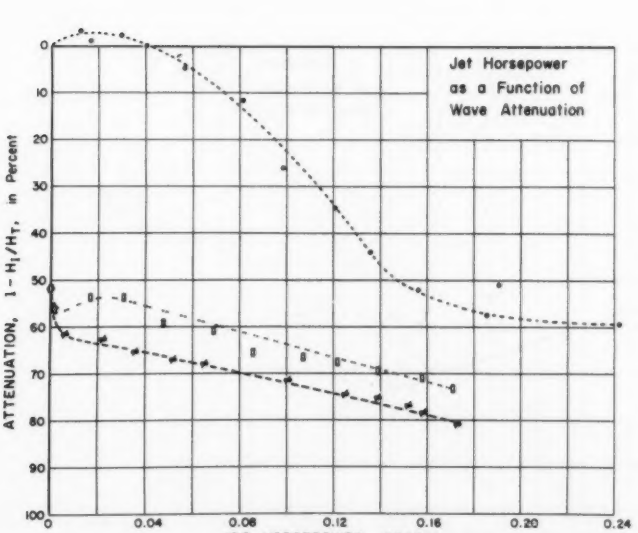
Water depth: 1 foot

Jet diameter: 0.161 inch
Total jet area: 0.00325 foot²

L/D: 6.16
Wave period: 1.25 seconds
Wave height: 0.21 foot

Reef width: 3.00 feet
Reef height: 0.70 foot

without reef
jets at SWL
jets 1" below SWL



HYD-783

FIGURE 19

best results that were obtained. These tables were computed to show the results that might be expected for a prototype installation in water 50 feet deep (Table II) and in water 100 feet deep (Table III).

The horsepower was computed for a 1000 ft. length of breakwater using the equation $HP_R = D_R^{5/2} \times 1000$, and the jet discharge was computed by the equation $Q_R = D_R^{3/2} \times 1000$. HP_R is the ratio of the prototype horsepower to model horsepower; Q_R is the ratio of the prototype jet discharge to model jet discharge; D_R is the ratio of the prototype water depth to model water depth.

For a 50 foot water depth the prototype wave would be 300 ft. long and 10.5 ft. high. If the height of this wave is attenuated 50%, the power for jets acting without a reef would be 2,700,000 horsepower and the jet discharge would be 26,000 cubic feet per second for 1000 ft. of breakwater. In the case of a submerged reef 35 ft. high and 33 ft. wide, acting in conjunction with the jets, the power required would be 1,200,000 horsepower and the jet discharge required would be 21,000 cubic feet per second for 1000 ft. of breakwater. This would be a reduction of over 50% in power, but the power requirement still would be extremely high.

For a 100 ft. water depth, the prototype wave would be 600 ft. long and 21 ft. high. In order to attenuate the height of a wave 50%, the power required in the case of the jets acting without a reef, would be 15,000,000 horsepower, and the required jet discharge would be 73,000 cubic feet per second. In the case of a submerged reef 70 ft. high and 67 ft. wide, acting in conjunction with the jets, the power required would be 6,600,000 horsepower and the jet discharge required would be 61,000 cubic feet per second.

If a reef which had the dimensions of 70% of the water depth in height and 300% of the water depth in width was used, it would not be necessary to use any jets to obtain the 50% attenuation.

From the above figures, it appears it would be best to build an artificial reef in such a way that it would be unnecessary to use any jets to obtain a reasonable attenuation. The cost to operate a hydraulic breakwater would be enormous. Provided power could be obtained at two cents per horsepower hour, the cost to operate a hydraulic breakwater without a reef, (which would cause a wave height attenuation of 50% for a wave 300 ft. long and 10.5 ft. high in 50 ft. of water), would be $47,000 per hour per 1000 ft. of breakwater. In the case of a reef 35 ft. high and 33 ft. wide, acting in conjunction with the hydraulic breakwater, the cost would be $24,000 per hour per 1000 ft. of breakwater.

If the efficiency of the breakwater could be increased to nearly 100%, (i.e. the same amount of power that was dissipated from the wave), the power required to attenuate a wave 50%, would be 46,000 horsepower for the 1000 ft. of breakwater for the 300 ft. long by 10.5 ft. high wave in 50 ft. of water.* The cost of operating such a breakwater would be $940 per hour per 1000 ft. of breakwater.

It would be necessary to increase the efficiency of the present breakwater before it would become feasible to introduce its use for general purposes.

*See Appendix I for wave power equation.

TABLE II

Prototype Horsepower Prediction

50 Ft. Water Depth
Reef Height 35 feet

300 ft. Wave Length
10.5 ft. Wave Height

Attenuation Percent	Jets without Reef		Jet with 35 ft. x 33 ft. Reef		Jet with 35 ft x 150 ft Reef	
	HP/1000 ft.	Q/1000 ft.	HP/1000 ft.	Q/1000 ft.	HP/1000 ft.	Q/1000 ft.
10	1,300,000	21,000	0	0	0	0
20	1,700,000	23,000	0	0	0	0
30	2,000,000	24,000	650,000	17,000	0	0
40	2,300,000	25,000	1,000,000	20,000	0	0
50	2,700,000	26,000	1,200,000	21,000	0	0
60	4,200,000	30,000	1,300,000	22,000	90,000	7,000
70	not obtainable		1,500,000	23,000	1,400,000	23,000
80	" "		2,100,000	25,000	3,000,000	28,000
90	" "		not obtainable		not obtainable	
100	" "		" "		" "	

TABLE III

100	"	"	"	"	"	"

TABLE III

Prototype Horsepower Prediction

100 Ft. Water Depth
Reef Height 70 feet

600 ft. Wave Length
21 ft. Wave Height

Attenuation Percent	Jets without Reef		Jets with 70 ft. x 67 ft. Reef		Jets with 70 ft. x 300 ft. Reef	
	HP/1000 ft.	Q/1000 ft.	HP/1000 ft.	Q/1000	HP/1000	Q/1000 ft.
10	7,000,000	60,000	0	0	0	0
20	9,500,000	65,000	0	0	0	0
30	10,400,000	67,000	3,800,000	48,000	0	0
40	13,000,000	71,000	5,700,000	56,000	0	0
50	15,000,000	73,000	6,700,000	61,000	0	0
60	24,000,000	85,000	7,600,000	63,000	500,000	20,000
70	Not Obtainable		8,600,000	65,000	8,000,000	65,000
80	"	"	12,000,000	71,000	17,000,000	80,000
90	"	"	Not Obtainable		Not Obtainable	
100	"	"	"	"	"	"

CONCLUSIONS

1. Shallow water waves in the laboratory studies can be attenuated by the use of hydraulic jet induced currents; however, the power required to attenuate such waves is relatively high, and the cost to operate such a breakwater would be prohibitive in a prototype installation.

2. The large power and flow rate required for a prototype installation would make such an installation almost physically impossible to construct.

3. The use of the combination of jets with a submerged reef increases the efficiency of the jets; however, the increased efficiency obtained would not justify combining the jets with a submerged reef to form a practical breakwater system, except possibly as a wartime emergency.

4. For a practical type of hydraulic breakwater, the energy required to attenuate a wave should be the same order of magnitude as the energy lost by the wave.

ACKNOWLEDGMENTS

The author wishes to express his appreciation to Professors J. W. Johnson and W. Littman and Mr. R. L. Wiegel for their helpful technical and editorial comments. The author wishes to thank Miss M. M. Lincoln for her assistance in preparing the figures and Mrs. V. Bauer for typing the manuscript. This work was done for a M. S. thesis in Mechanical Engineering at the University of California. This work was partially supported by Contract Nonr-222(46) with the Office of Naval Research.

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Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

SAND BY-PASSING PLANT AT LAKE WORTH INLET, FLORIDA

Charles Senour,[1] M. ASCE, and John E. Bardes,[2] J.M. ASCE

Background Data

Lake Worth Inlet, Florida, provides access to the Port of Palm Beach, on the Atlantic coast some 70 miles north of Miami. The project channel is 300 feet wide between rubble stone entrance jetties 900 feet apart, concrete-capped to elevation 7 feet above mean low water. The latter is the datum to which elevations and depths are referred herein unless otherwise indicated. The inlet and jetties were constructed by local interests between 1918 and 1925. Maintenance was taken over by the Federal Government in 1935. Project depth is 25 feet, with increase to 35 feet under study. Actual depths are in excess of 27 feet. The channel bed is in rock. The north jetty is 2,000 feet long and the south jetty 1,900 feet, both measured from the 1920 shoreline. They were carried to the minus 20-foot contour when constructed. Vessel transits exceed 30,000 annually. In the seaward 1,000 feet or so of the channel, ocean swells are practically undiminished.

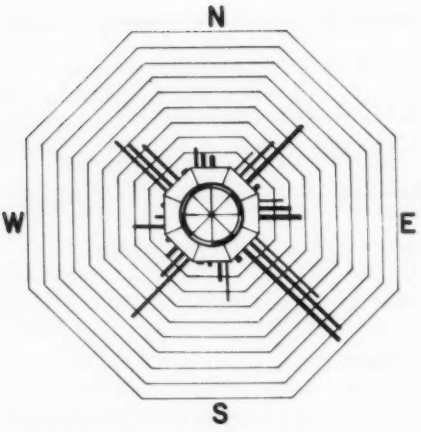
The mean tidal range is 2.3 feet, and the spring tide range 3.3 feet. The lowest tide expected is 2 feet below m.l.w. The maximum tide of record was 11.2 feet during the hurricane of September 1928, and the second was 8.7 feet during the July hurricane of 1936.

Maximum wave heights are believed to result from hurricanes tracking north to northwestward outside the Bahama Islands. In connection with Hurricane Connie in 1955, ground swells from the northeast attained an observed height of 18 feet a short distance northeast of the seaward end of the north jetty. They broke in that area, but surf ran over several hundred feet of the seaward end of the jetty at estimated elevations of 12 feet maximum, and 10 feet average. This was considered a severe condition, but possibly not maximum.

Average current velocity in the jettied channel at strength of flood and ebb tides is about 5 feet a second. Occasional extreme tides of 6 feet or so step the figure up to a little over 5 knots. Longshore currents, measured in the summer of 1939 in shallow water, during a period of northward drift,

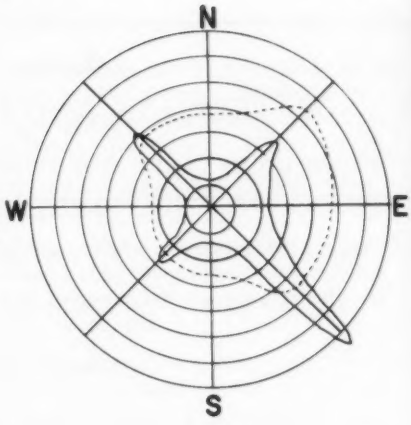
Note: Discussion open until August 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1980 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 1, March, 1959.

1. Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.
2. Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.



AVERAGE DIRECTION, DURATION AND VELOCITY OF WINDS FOR ONE YEAR AT WEST PALM BEACH, FLORIDA.

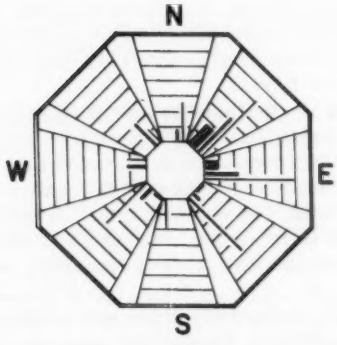
VELOCITIES MPH
0 TO 10
11 " 30
OVER 30



AVERAGE VELOCITY, DURATION AND MOVEMENT OF WINDS FOR ONE YEAR AT WEST PALM BEACH, FLA.

AVERAGE VELOCITY IN MPH .....
DURATION OF WIND IN DAYS _____

WIND DIAGRAMS



LOW SWELLS (1-6 FT.)
MEDIUM SWELLS (6-12 FT.)
HIGH SWELLS (OVER 12 FT.)

IN THE SWELL DIAGRAMS THE LENGTH OF THE BAR DENOTES THE PERCENT OF THE TIME THAT SWELLS OF EACH TYPE HAVE BEEN MOVING FROM OR NEAR THE GIVEN DIRECTION.

SWELL DIAGRAM

PLATE I

SCALE IN FEET

500 0 500 1000 1500

AERIAL VIEW
LAKE WORTH INLET

PLATE
2

were too low to produce erosion, but were capable of moving sand already placed in suspension. In deep water off-shore, northward currents were capable of causing erosion.

Littoral Drift

The beach material is about 40 percent silica sand and 60 percent shell fragments. It is light and unconsolidated, and easily moved. Swells from the southeast produce northward drift and those from the northeast cause southward movement. The latter predominates.

The wind diagrams shown in Plate 1 show the prevailing winds to be from the southeast, but the swell diagram shows a definite preponderance of swells from the northeast and east. Those from the east probably produce little movement either northward or southward. Heavy swells, generated far off-shore, with crests running NW-SE, probably cause the heaviest sand movements. They can, and often do, roll in in complete disregard of a fairly fresh contrary wind.

Effects of Lake Worth Inlet Jetties

Inasmuch as the north jetty lies directly across the path of the southward drift, it is inevitable that it should cause impoundment. The extent to which it has done so is strikingly shown by the aerial photograph reproduced as Plate 2. The beach north of the inlet is about 1,400 feet seaward of the beach down-drift of the south jetty. The impounding capacity of the north jetty is believed to have been reached, or nearly reached.

The Beach Erosion Board of the Corps of Engineers, U. S. Army, found, from a cooperative study of survey scope undertaken in 1955, that between 1929 and 1955 the average rate of accretion shoreward of the minus 100-ft. contour within a distance of 2-1/2 miles north of the inlet was about 230,000 cubic yards a year. The figure includes the deposited material removed by maintenance dredging from the jettied channel and from the inner bar as well as that which has widened the beach and formed a small submerged delta near the inlet. During the same period, an average annual net loss of 66,000 cubic yards a year was suffered, landward of the minus 30-ft. contour, by the 2-1/2 miles of beach immediately south of the inlet. For the 15-1/2 miles or so between the Lake Worth Inlet and the South Lake Worth Inlet, the average annual loss was estimated at about 450,000 cubic yards, of which 200,000 cubic yards occurred shoreward of the minus 18-ft. contour, and 250,000 were lost between the minus 18-ft. and the minus 30-ft. contours. In general, the soundings south of the inlet did not extend beyond the minus 30-ft. contour. The profile north of the inlet showed scours and fills of 10 to 15 feet at depths of 80 and 85 feet between 1929 and 1955. Fill at the several contours investigated was not always continuous updrift of the inlet, nor was scour continuous below it. The figures given are the algebraic sums of local fills and scours.

Corrective Measures Attempted

The starving of the beaches south of the inlet naturally caused alarm. Restoration by means of groins was attempted, but, on the whole, proved

unsatisfactory. In 1944, as an alternative to groins, a stockpile of sand containing about 280,000 cubic yards was pumped from Lake Worth onto the beach at a point about 1,500 feet south of the south jetty. Observations four years later, in 1948, disclosed that the stockpile, distributed by the surf, had benefited the beach for a quarter of a mile to the north and for a mile and a half to the south.

In 1947 the Beach Erosion Board recommended that a million cubic yards of sand be pumped from Lake Worth and deposited on the beach in five stockpiles spaced some 4,000 to 6,000 feet apart—the northernmost to be at Eden Road, about 9,000 feet south of the inlet. In 1948 and 1949, pursuant to this recommendation, some 2-1/4 million cubic yards, and in 1953 about 463,000 cubic yards additional, were placed in stockpiles. In 1953, in a staff report to the Engineer, Town of Palm Beach, the Beach Erosion Board observed that most of the stockpiled material had moved southward and had benefited the shore for a distance of five miles or more south of the inlet; that the beach for about 10 miles south of the inlet apparently needed nourishment; and that the evidence afforded by the 1948 nourishment operation indicated that at least 10 miles of shore could be adequately protected by periodic nourishment at a rate of about 200,000 cubic yards a year.

Selection of Remedy

The lack of success of groins and the encouraging results of artificial nourishment left little doubt as to the nature of the proper remedy, but considerable question remained as to the method of its application.

At the time that Tippetts-Abbett-McCarthy-Stratton were called in as consultants, in 1954, there was some doubt as to the extent of sand deposits available in Lake Worth. One estimate as low as a million cubic yards had been advanced. The 1955 study of The Beach Erosion Board had resolved all fears of scarcity. Apparently, there are more than 80 million cubic yards available in the lake. Thus, it apparently would be possible to replenish the beaches for something like 180 years at 450,000 cubic yards a year, from that source alone. The lake sand, however, is finer in texture than the beach sand, is of different color, and often shows impurities in the form of black earth, high in organic matter, which, until it has been exposed to the surf for a time, is somewhat objectionable. It was felt that it would be desirable to replenish the beaches with beach sand if and to the extent that it was practical to do so.

Use of a pipe line suction dredge in the open ocean patently was out of the question. Some thought was given to the use of a hopper dredge excavating from off-shore sources and pumping from its hoppers onto the beach while moored to the south jetty. Investigation with owners of hopper dredges developed a probable cost of $2.00 a yard on the basis of an annual stockpile of 200,000 cubic yards,—assuming the availability of offshore sources, about which, because of the presence of coquina ledges, there is a measure of doubt.

Again assuming an adequate off-shore source of sand supply, it would be possible to replenish the beach by means of an L.C.T. or L.S.T. converted into a hopper dredge capable of pumping both into and out of its hoppers, but a rough calculation showed that costs again would be excessive.

This focused consideration on a stationary pumping plant at the north jetty.

The practicality of a fixed installation had been demonstrated in principle at the South Lake Worth Inlet, some 15-1/2 miles downdrift, previously mentioned herein.

The South Lake Worth Inlet was dredged in 1927 to induce a circulation of water through Lake Worth. It is a shallow channel, 125 feet wide, accommodating 6- to 8-ft. draft vessels. Its entrance jetties, 310 to 320 feet long, reach to the minus 6- or 8-ft. contour. From 1931 to 1936, sand passing into Lake Worth through the inlet caused shoaling in the lake at the rate of 165,000 cubic yards a year. By 1936 the north jetty's capacity for impounding drift had been reached, and in 1937 a small by-passing plant was constructed about 50 feet inshore from its seaward end. The pump suction excavates a trench 8 or 10 feet deep to hardpan or rock, about 30 feet long, roughly normal to the shore line, and reexcavates as long as sand keeps filling it. In quiet weather very little pumping is required. In rough weather 18 hours a day is sometimes insufficient, and on occasion, it is said, the trench and even the inlet channel have been filled despite the pump. Its 8-inch discharge line runs along the jetty to shore, crosses the inlet on the highway bridge, and discharges on the south beach. Capacity is estimated at about 76 cubic yards an hour, and annual output at about 80,000 cubic yards.

Under this amount of replenishment, the south beach has prospered. If the annual volume of southward drift is of the order of 230,000 cubic yards at the South Inlet, as it appears to be at the main Lake Worth Inlet the shoaling of the lake at the yearly rate of 165,000 cubic yards through a channel 6 to 8 feet deep (in absence of by-passing plant operation) would imply that the bulk of the drift reaching the beaches to the south of the south inlet took place above the minus 6- to 8-ft. contour. Pumping less than half of that quantity across the inlet apparently was enough to maintain the south beaches fairly well. The implications were encouraging, but the condition probably results from hydrographic and hydraulic conditions peculiar to the specific locality.

Problems Peculiar to Lake Worth Inlet

The situation at the Lake Worth Inlet differs in several respects from that at the South Inlet.

The Lake Worth Inlet jetties presently end in about 15 feet of water and are consequently exposed to high swells. The inlet is 900 feet wide; the navigation channel has a minimum depth of 27 feet, and there is no bridge. The public easement along the north jetty is about 110 feet wide, and the beach owners to the north are opposed to any operation that would, or even could, withdraw sand from the accretion already made, or cause noise audible above that of the surf, or be housed in a structure unacceptable esthetically.

The prohibition of noise dictates electric, in lieu of diesel, power; and the exposed location of the jetty rules out a plant site closer than 100 feet to its seaward end. Since the noise of sand passing through metal pipe would be objectionable, the pipe line where not submerged must be buried. Esthetic considerations suggest a plant structure combining some degree of architectural beauty with the great strength required to resist seas.

One of the greatest problems was, of course, how to convey the sand from the north bank to the south bank of the inlet in absence of a bridge to support the pipe line. A floating discharge line would be impractical because of the density of vessel traffic, even if a means of avoiding rupture from swells

could be devised. Since construction of a pipe line bridge with sufficient clearance seemed out of the question, the use of a submerged line was indicated.

Precautions Against Plugging

This immediately raised the spectre of plugging. The Government had ruled that the crossing line must be in a trench at least three feet deep measured from the bed of the navigation channel. Inasmuch as such a trench would promptly fill with sand, the operation of raising, unstopping, and replacing a plugged line seemed so formidable as to warrant unusual precautions against plugging. The use of electric power to drive the pump introduced the hazard of plugging resultant from sudden power failure as well as from other causes, such as overloading the line or choking of the suction intake by seaweed, sand, coral or foreign matter.

In the plan as originally recommended by the consultants, the following safeguards against plugging were suggested:

A. Installation, near the operator's gauges, of a device which will emit an audible alarm whenever vacuum in the suction line rises above a pre-set value; whereupon the operator will immediately raise the suction and pump clear water. (If the suction head were choked off by seaweed, raising the suction might not suffice to correct the situation.)

B. As an added precaution, therefore, installation in the suction line of an automatic device which, upon a rise in vacuum beyond a predetermined value, will instantly and automatically admit clear water to the suction side of the pump.

C. Installation at the north end of the submerged line of a motor-driven air compressor and of a flush tank containing, under sufficient pressure to produce an initial velocity of 16 feet a second in the submerged line, a quantity of water equal to about twice the volume of the submerged line; the tank to feed clear water into the submerged line automatically upon failure of power to the main pump, the velocity of flow decreasing, of course, with emptying of the tank and expansion of the air.

D. Installation adjacent to the flush tank of a 12-inch emergency centrifugal pump to pump clear water from the inlet through the submerged line with a velocity of 12 feet a second; the emergency pump to be driven by an internal combustion engine starting automatically from battery upon failure of power to the main pump. It was considered that the emergency pump would easily attain speed by the time the flush tank had emptied. It was recommended that provision also be made to enable the operator to start the emergency pump from the main control room, and that he be required to operate it briefly at the beginning of each shift, to insure its proper functioning.

Both flush tank and emergency pump were to be provided with necessary check valves to prevent flow elsewhere than through the submerged line, and to prevent air flow to the latter.

It was intended that when current had been restored following a power failure, the main pump would be started slowly, pumping clear water until the whole line was flushed out. Unless closure of the check valve to prevent backflow had caused the line to plug between main pump and check valve, the

situation would be quickly restored to normalcy. If the shore line had become plugged, the check valve would remain closed, and the auxiliary pump would be continued in operation until the line was restored to service—or at least until all danger of plugging the submerged line had been overcome. It was assumed that an internal combustion-engine-driven emergency stand-by pump would be permissible and that its operation for a few minutes at the beginning of each day's pumping, as a check upon its readiness to serve, would not be considered objectionable by local property owners.

E. The procurement and on-site storage, as a measure of last resort, of a self-propelling nozzle of the type used in opening blocked sewers. Such a device is fed water through a fire hose and digs its way through the obstruction, the material it removes being carried out with the effluent water. It was assumed that fire hose and a portable water pump to supply the necessary pressure could be provided by the Town in event of need.

Capacity of Plant

Inasmuch as the total of southward drift reaching the inlet area is estimated to be of the order of only 230,000 cubic yards a year, the annual output of the plant does not have to be very large. But the rate at which sand will reach it is both variable and unknown. Experience at the South Lake Worth Inlet seemed to be the best criterion. The plant at that locality, with a capacity of 76 cubic yards an hour, had apparently managed to pump about a third of the assumable total volume of annual drift, and less than half the volume moving above the minus 6- to 8-ft. contour. Its capacity was too small to handle material as rapidly as it arrived when movement was especially heavy, and its excavated trench had very little storage capacity to compensate for deficiency in pump capacity.

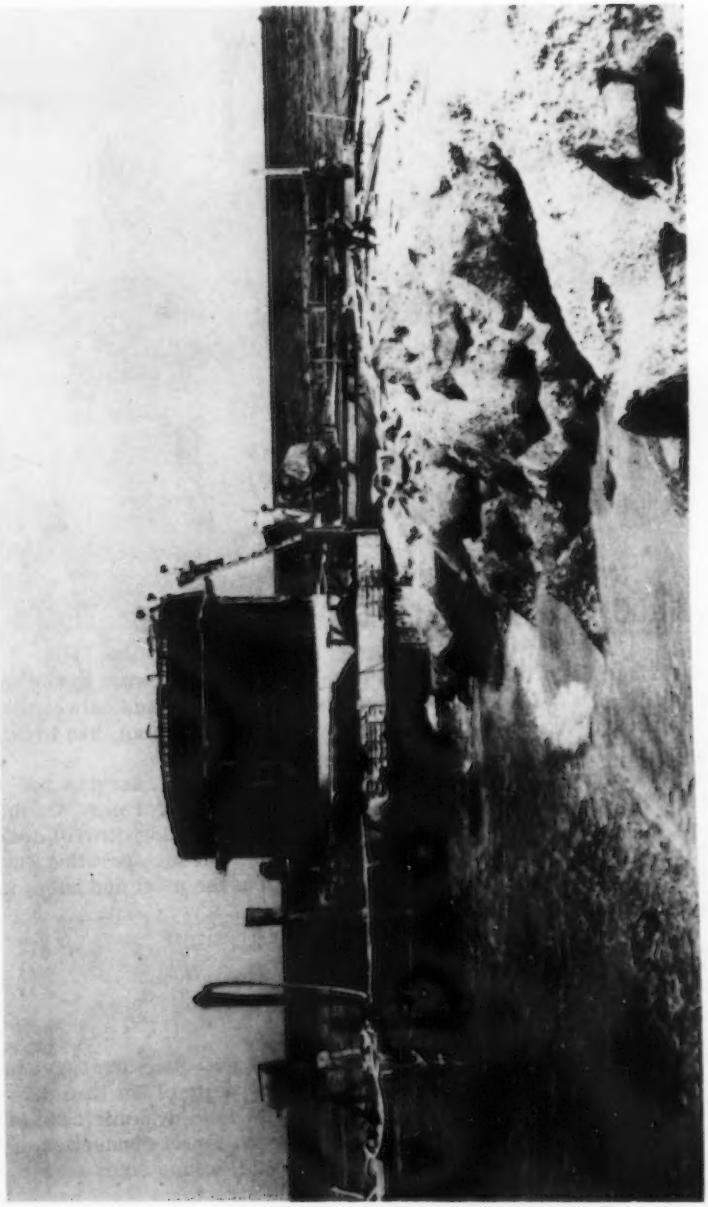
The obvious remedy, if provision were to be made for pumping the entire volume of drift, as seemed desirable, was increased pump capacity and, if possible, increased temporary storage. The greater both were, the more likely would be the plant to handle everything that came to it. A pump capacity of 250 cubic yards an hour was recommended as a compromise between what was probably needed for complete ability to cope with heavy drift and what could be afforded. A 15-inch suction, 12-inch discharge, and 16 feet/second line velocity were proposed. Cost estimates ran so high, however, that modification in capacity and omission of some of the suggested accessories were necessary.

As-Built

Pump House

(See Plates 3 and 4.) The structure housing the pumping equipment is a heavily reinforced monolithic concrete building, elliptical in plan, with major axis parallel to the jetty. Inside dimensions are 18 feet by 40 feet. Walls are one foot six inches thick. Openings through which pipes pass are provided with specially designed devices for watertightness, and all windows and door openings are equipped with watertight steel shutters for use during severe storms.

The structure is supported on a reinforced concrete caisson founded on



PUMPHOUSE - SHOWING REINFORCEMENT

PLATE
3

Plate 4. **COMPLETED STRUCTURE**

sand at elevation minus 28.0 feet. The lower part of the caisson has been sealed by means of a 3-ft. tremie concrete slab, and the space between the slab and the main floor of the plant, at elevation minus 1.0 foot, has been filled with beach sand to increase lateral stability.

The main floor (see Plate 5) houses the pump and motor, service and gland water pumps, sump pump, and power-supplying transformer. On the upper floor, elevation 11.5 ft. (see Plate 6), are the operating control desk, motor controllers, ventilating equipment, and equipment for operating suction hose boom and mast. The sheave sector for slewing the mast and boom is on the reinforced concrete roof at elevation 24 ft.

Main Equipment (See Plate 6)

The Dredge Pump

The dredge pump has a 12-inch suction inlet and a 10-inch discharge outlet. It has been designed for a rated clear water capacity of not less than 4,000 G.P.M. at approximately 690 R.P.M. against a total dynamic head of 198 feet. The operating characteristics are based on direct connection, through a flexible coupling, to a 400 H.P., 720 R.P.M. wound rotor motor. Motor and pump are mounted on a common welded structural steel bedplate.

The pump impeller is enclosed and rotates in a volute type casing, equipped with replaceable liner plates. All wearing parts are of an abrasion-resisting steel alloy. The pump specifications require it to handle up to 15 percent solids at an efficiency of not less than 60 percent. The rate of sand

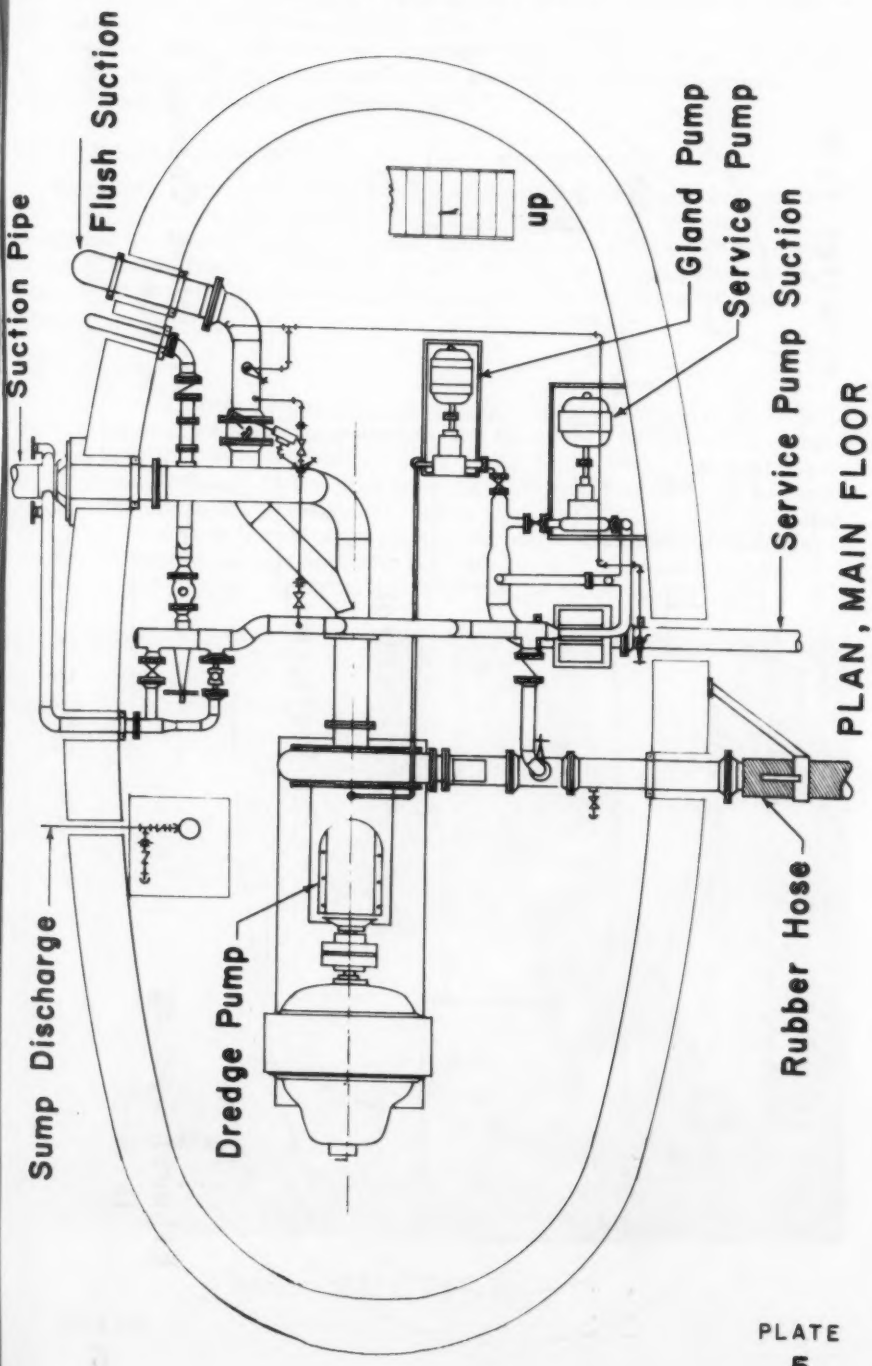

Flush Suction
Suction Pipe
up
Gland Pump
Service Pump
Service Pump Suction
PLAN, MAIN FLOOR
Sump Discharge
Dredge Pump
Rubber Hose

PLATE
5

Water Tank
down
Air Intake Opening In Roof
Duct Opening In Floor
Control Panel
Main Pump Motor Control
Fan Platform
Exhaust Opening In Roof
PLAN, UPPER FLOOR

PLATE 6

transfer, under such conditions, will be about 175 cubic yards of saturated sand an hour. Pipe line velocity will be 16 feet a second.

A vacuum relief valve has been provided in the suction line just ahead of the pump, which will automatically admit clear water whenever intake vacuum exceeds 23 inches of mercury.

Power Supply Transformer

The power supply transformer is a double winding, 3-phase, dry type and is rated at 112-1/2 KVA providing a primary voltage of 4,160 volts. The power cables are carried within steel, zinc-coated conduits which have been encased in concrete for protection against the elements. All possible measures have been taken to prevent objectionable radio and television interference.

Suction Line

(See Plate 7)—The suction line consists of a horizontal limb of 12-inch rubber hose suspended beneath a boom, and joined by a 90-degree pipe angle to a vertical limb also of 12-inch rubber hose. The vertical limb terminates in a bell-shaped steel suction with trash bars. Two three-inch jets are provided. The length of the horizontal limb is such as to bring the suction head to within three or four feet of the edge of the public easement north of the jetty when digging at right angles to the jetty. It will excavate a kidney-shaped trench to about elevation minus 15 feet, at which approximate

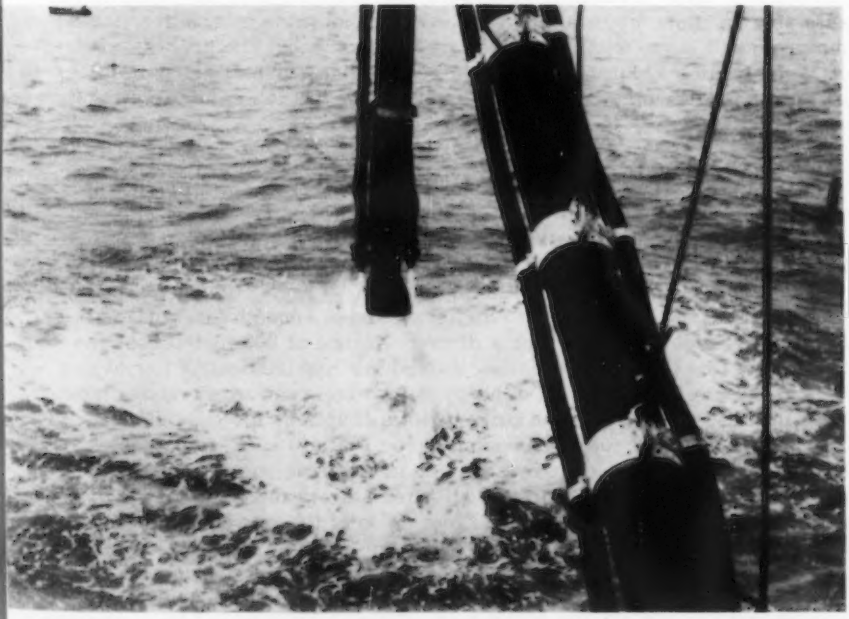


Plate 7. **SUCTION**

WW 1

elevation material too firmly cemented to be pumped is encountered. Thus the trench will be as long and as deep as easement limitations and geologic conditions permit.

Hoisting Equipment

The hoisting equipment consists of a mast and boom arrangement which carries the 12-inch suction hose while transporting it throughout the pumping area. The boom arrangement has been designed to control both horizontal and vertical movement over the entire pumping range. Horizontal movement is controlled by a slewing or rotating assembly which will turn the boom through an arc of 140 degrees at a maximum periphery speed of 50 feet a minute.

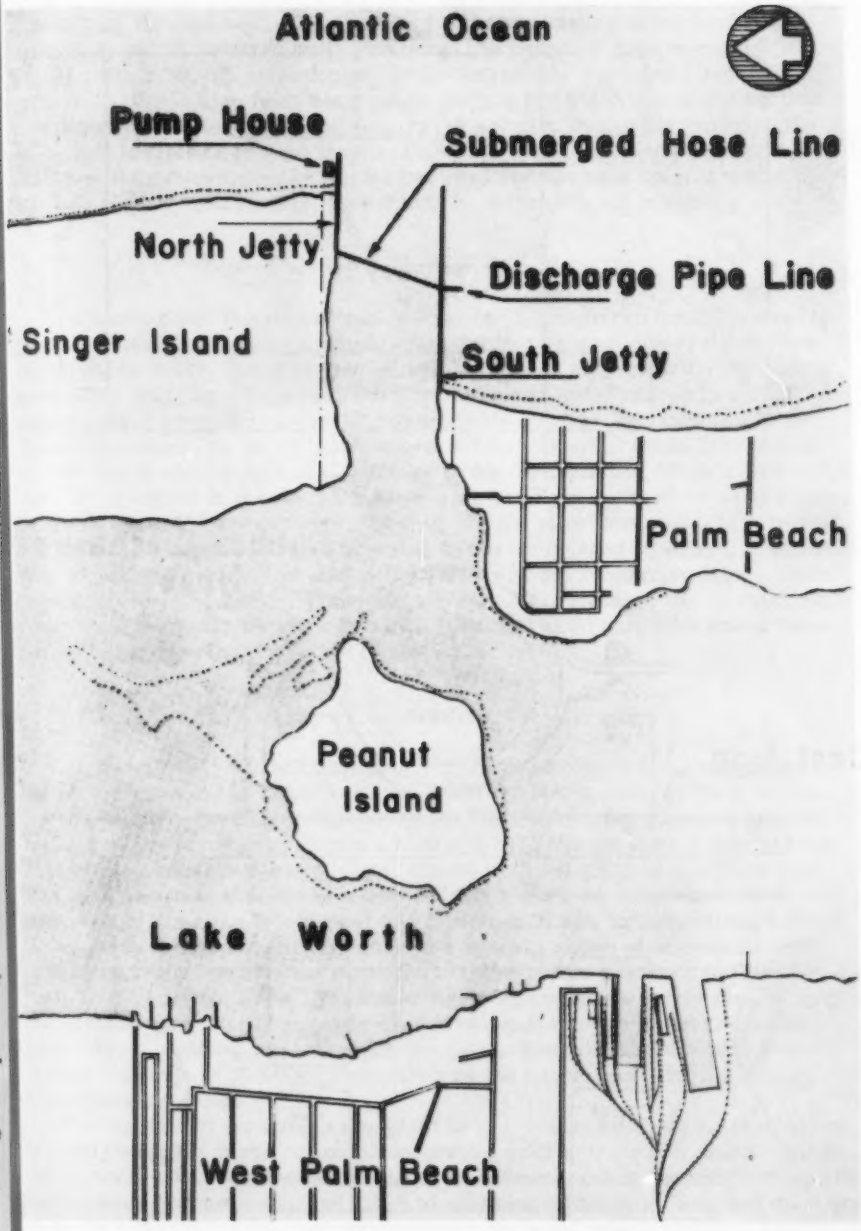
The luffing or boom-raising-and-lowering assembly is capable of raising the fully loaded boom through a vertical arc of 25 degrees in approximately one minute. Two mooring posts have been provided at the limit of westward travel to permit securing of both the boom and the hose during rough weather or inoperative periods.

Protective Groin (See Plate 8)

The pump house with its suction hose boom lies completely within an area bounded by an "L" shaped groin to the west and north and by the north jetty to the south. The primary purpose of the groin is to maintain the beaches to the north in their present or "pre-pumping" condition. In this respect, it acts as a bulkhead or retaining wall. Only sand which moves over the top of the groin will come within range of the suction head. The groin is made up of 34-ft. lengths of interlocking steel sheet piling driven flush with the existing beach profile and capped. Posts erected on the cap carry a safety line marking off the area to the public.

Discharge Line

Between the pump house and the jetty, the 10-inch discharge line is of wire-reinforced rubber hose. It makes a 90-degree bend at the jetty face, changes from rubber hose to steel pipe of 1/2 inch wall thickness, and extends thence 700 feet parallel to the jetty, buried beneath the sand, to the point of the inlet crossing. For a distance thence of 800 feet across the floor of the inlet (the Government having waived the requirement of burial in a trench) the line is comprised of heavy duty, wire-reinforced rubber hose, made up in 50-ft. lengths with built-in steel couplings for easy disconnection. It is held in place by cast iron anchors (see Plate 9) spaced 10' -10" apart. The hose is of the smooth bore type, and is constructed of pure gum rubber protected by multiple layers of duck. After emergence from the water at the south side of the inlet, the final 100 feet or so of the line consists of 1/2-inch-thick steel pipe. As presently built, the line will discharge at the south face of the south jetty, and the material will lie within the shadow of the latter. The total length of discharge line is 1,750 feet. The consultants had suggested discharging beyond the shadow of the jetty but, upon representation of local interests that prior experience with a stockpile placed adjacent to the jetty and been satisfactory, had concurred in a compromise location for the



SITE PLAN

PLATE 8

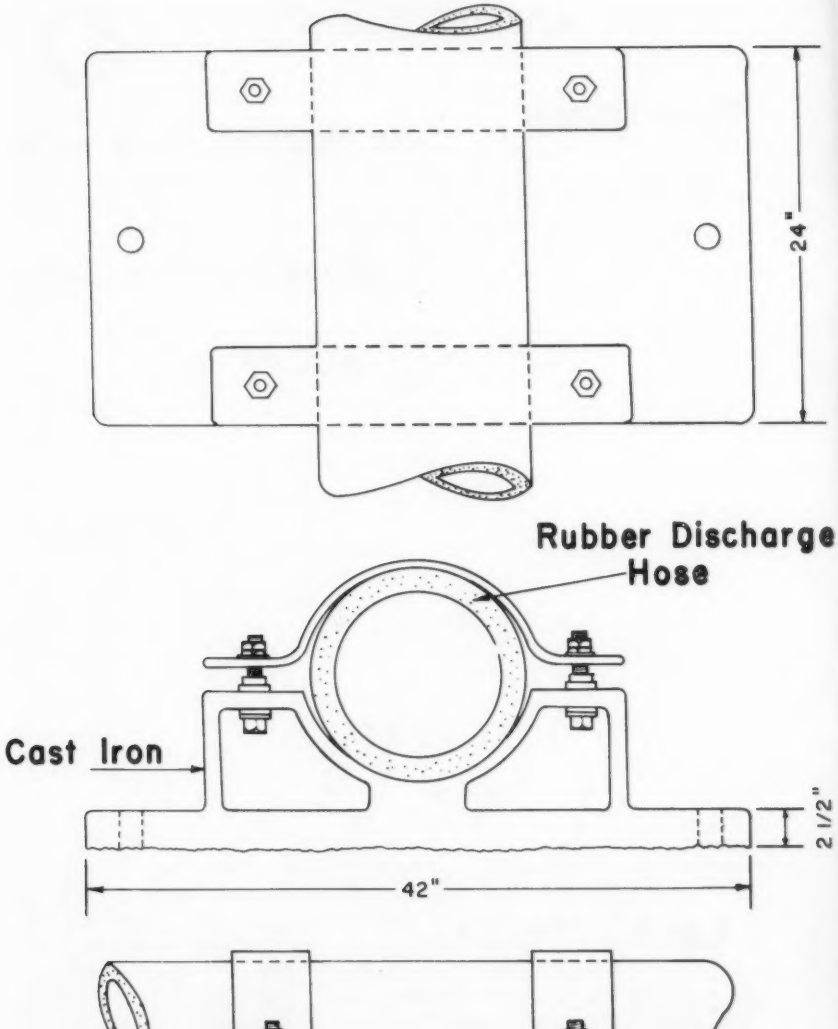


PLATE 9

ANCHOR - SUBMERGED LINE

point of discharge—namely, adjacent to the south jetty, about 1,400 feet inshore from its outer end. The line as finally built reaches the south jetty 950 feet from its seaward end. The pump has been designed to pump 3,900 G.P.M. with a 300-ft. extension of the discharge line southward beyond the jetty face. It has also been designed to pump 3,860 G.P.M. with the suction line extended 100 feet seaward, i.e., to the seaward end of the jetty. In the event that it later should develop that a worth-while additional amount of drift would be intercepted by such a seaward suction assembly, it would be feasible to pump alternately from it and from the present assembly.

Line-Flushing Equipment (See Plate 10)

In the event that the submerged portion of the discharge line threatens to become clogged with sand due to a power failure or stoppage of the dredge pump, clean water, under pressure, will be flushed automatically through it to prevent plugging. Water for the flushing operation is stored in a 17,000-gallon tank supported in a reinforced concrete cradle. The tank has been designed to hold sea-water under a pressure of 70 psig. On failure of the power supply or stoppage of the dredge pump, or reduction below a pre-set limit of pressure in the discharge line near the flush tank, a normally closed solenoid-operated 4-way valve will open the air pilot line, which in turn will open a butterfly valve, releasing water to the submerged pipe line. The water content of the flush tank is double the volume of the submerged line. Air required to close the butterfly valve at the end of the flushing run is supplied from a separate air receiver in which it is held at 66 psig by a check valve during the pressure drop in the flush tank.

Discussion

The survey data of 1883 did not cover the southernmost several miles of the shore between the Lake Worth Inlet and the South Lake Worth Inlet, but, for the portion covered, comparison with the 1929 survey shows in general that accretion occurred. Between 1929 and 1955 the net change was erosion of the beach between the two inlets, except in the vicinity of the South Inlet. The implication is that the loss between 1929 and 1955 is attributable to the presence of the Lake Worth Inlet and jetties, although its magnitude, 450,000 cubic yards a year, is almost double the annual volume of southward drift prevented from reaching the southern beaches by the presence of the inlet, viz., 230,000 cubic yards. The latter figure corresponds fairly closely with the 200,000 cubic yards annually eroded between the two inlets above the minus 18-ft. contour, and also with the Beach Erosion Board's 1953 pre-survey estimate of 200,000 cubic yards as the annual quantity of artificial nourishment needed.

The function of the by-passing plant is to transfer as much as possible of the 230,000 cubic yards of annual southward drift to the south beach. In order to transfer the entire amount it would, assuming that the jetty's impounding capacity had been reached, have to excavate a trench so long and deep that it would cross all of the contours at which material was in motion. Length of trench is determined by length of suction arm. This has been made as long as the width of easement permits, and depth of trench as great as geologic conditions allow. How much of the drift it will intercept will have to be

Pipe Vent
Compressor House
Compressor
Air Inlet
Flush Tank
Air
Water
FLUSH TANK
Butterfly Valve

PLATE 10

determined by experience, which may indicate the installation of a second seaward suction assembly to be desirable.

It is quite possible that there will be periods of sand movement when the weather is too rough for plant operation. Rough surf, however, often deposits sand to high elevations well inland on the right-of-way, and no reason is apparent why such deposits could not easily be moved to within reach of the suction by mechanical means when weather conditions again become favorable. Indeed the maintenance of a trench parallel to the jetty in calm weather might provide enough temporary storage capacity to effect significant increase in annual output. Annual plant capacity greatly exceeds average annual volume of drift, of course. The Beach Erosion Board has estimated that 150,000 cubic yards will reach the plant annually and that, of this total, it will be able to transfer about 100,000 cubic yards to the south beach. It is hoped that the latter figure can be materially increased by proper vigilance and action.

Just what the effect of pumping will be with respect to the beaches downdrift is also unknown at this time. Several years of operation will probably have to precede the definite determination of benefits. However, the surprisingly favorable results at the South Inlet from small quantities of artificial nourishment and the visible effects of the stockpiling of 1948-9, which led to the conclusion that 200,000 cubic yards of annual artificial nourishment would suffice to replenish at least 10 miles of beach, suggest that the deposition south of the inlet of something between 100,000 and 200,000 cubic yards annually by the by-passing plant may alter appearances in the shallow waters seen and used, to a much greater degree than the mathematical ratio between annual losses and artificial replenishment of the magnitude envisioned for the by-passing plant would imply. Erosion below the minus 18-foot contour, which, if continued, would ultimately affect the visible and used portions of the beach, will have to be replaced by material from other sources,—probably Lake Worth. The ideal solution would appear to be to build the beaches at once to such elevations and widths as are deemed appropriate, by large-scale dredging from Lake Worth; to preserve to the greatest degree practical their most-used areas by beach sand pumped across the inlet; and to correct such future deficiencies as from time to time may still develop in the deeper off-shore waters—and perhaps inshore to a lesser extent—by periodic pumping from Lake Worth.

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OIL LOADING TERMINAL FOR PAKNING, SUMATRA

Barnett Silveston,[1] M. ASCE

SYNOPSIS

In 1957 a new oil loading terminal for N. V. Caltex Pacific Petroleum was completed in Sumatra, Indonesia. The terminal, designed to accommodate simultaneously two 50,000 deadweight ton tankers, consists of a wharf supported on steel piles in deep water about 1,800 feet offshore. A 22-inch diameter submarine pipeline delivers crude oil from the shore to the wharf.

This paper describes the construction of the wharf and pipeline, including the energy-absorbing fender system and means employed to protect steel in the structure against corrosion.

INTRODUCTION

It is now well known that during recent years there has been a most remarkable increase in the size of ocean-going petroleum tankers. During World War II, the largest tankers afloat were the T-2's of 16,700 d.w.t. carrying capacity with a length of about 524 feet and full draft of about 30 feet. Now, tankers of 45,000 d.w.t. capacity are comparatively common and several giant tankers of 85,000 and 100,000 d.w.t. with lengths up to 950 feet and drafts of 48 feet are already in operation or under construction.

With the advent of these large carriers has arisen the problem of providing adequate docking facilities at marine terminals. This problem is aggravated by the fact that very few harbors in the world have sufficient depths of water for the largest vessels. Since the cost of dredging to provide deep water in existing harbors is generally prohibitive, it is often necessary to resort to unconventional methods of providing the necessary facilities. Thus, offshore mooring structures in deep water served by submarine pipelines

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1. Partner Tippetts-Abbett-McCarthy-Stratton, Engrs., New York, N. Y.

from shore offer a solution to the problem. One such terminal is that constructed for N. V. Caltex Pacific Petroleum Mij. in Sumatra.

Pakning Terminal, Sumatra

N. V. Caltex Pacific Petroleum Mij. exports crude petroleum from the Minas oil fields in Central Sumatra, Indonesia. Petroleum is transported from the oil fields by river tankers down the Siak River to Pakning on the east coast of Sumatra. At Pakning, the oil is stored in tanks from which it is loaded into ocean-going tankers for transport to the United States.

In 1953, a wharf was constructed at Pakning to accommodate tankers of 28,000 d.w.t. In 1955, as part of a major expansion program, N. V. Caltex Pacific Petroleum Mij. decided to provide a second wharf capable of accommodating the large tankers being planned at that time. This wharf was completed in March, 1957 and, although designed for 50,000 d.w.t. tankers, tankers of 85,500 d.w.t. have been successfully loaded there.

General Description of Wharf No. 2

The new wharf is located about 1,800 feet offshore in about 60 feet of water. The range of spring tides is about 10 feet and tidal currents frequently run at 3.5 knots. One berth is provided on each side of the wharf which is designed to accommodate two 50,000 d.w.t. tankers simultaneously.

The wharf, as shown on Plate I, consists of a central loading platform 180 feet long and 40 feet wide with intermediate and end mooring dolphins on each side. The two intermediate dolphins are connected to the central platform with walkway bridges 104 feet long. The end mooring dolphins are isolated structures, launches being used to handle ships' bow and stern lines at these points. The central platform and the dolphins are timber decked steel structures supported on steel pipe piles. Energy-absorbing fender panels are installed on the central platform and the intermediate dolphins. A 22-inch diameter submarine pipeline connects the wharf with shore installations.

Piles

The river bottom at the wharf site consists of several feet of loose sand overlying a thick stratum of clay, which is soft near the surface but becomes firm with increasing depth. The maximum pile loads were estimated to be 70 tons in compression and 55 tons in tension. To mobilize adequate bearing capacity in the clay, vertical piles were to be 140 feet long and batter piles 150 feet. Steel pipe piles of 20-inch diameter and 1/2-inch wall thickness with closed ends were selected as the most suitable type of pile.

Steel pipe in lengths of 40 feet with edges bevelled for welding was purchased in Germany and delivered to fabricating shops in Singapore. The finished piles were then transported by barge across the Strait of Malacca to Pakning, a distance of about 250 miles.

In order to pitch each pile, holes were drilled near the closed end allowing the tip to sink. The head of the pile was lifted by a floating crane and set in the leads of a floating pile driving rig. The pile was held in position against currents by a guy line, attached near the tip of the pile. The guy line was

ISLAND TYPE WHARF AT PAKNING SUMATRA

PLATE
I

removed immediately before driving. Piles were driven by a 15,000 ft.-lb. single acting steam hammer.

Sequence of Construction

The wharf is located in deep water and exposed to winds and tidal currents. To prevent damage to the piles by vibration or excessive deflection before the superstructure was completed, the engineers prepared a sequence of construction that would achieve stability of the various structures as the work progressed.

Construction of the breasting platform began with driving the central cluster of four batter piles. These piles were lashed together and tied to an anchored barge. The pairs of batter piles on each side of the structure were next driven, lashed together and tied to the central cluster by means of temporary bracing above low water level. The bracing provided a working platform for preparation of the pile heads. The piles were cut off to the correct elevation and slots were cut to receive the gusset plates connecting adjacent piles. Plate collars were slipped over the pile tops and fastened temporarily below the slots. Gusset plates were then inserted into the slots and welded, after which the collars were raised to fit against the gussets and tack-welded. Stiffeners were also tack-welded to piles and collars. The cap beam, consisting of 36 WF 150 section, was then tack-welded to the piles and the entire assembly finally welded.

While work progressed on the center bent, pile driving continued on the adjacent bents. After driving, the pile groups in the adjacent bents were tied to the center bent by temporary members. Pile tops were slotted and capped and longitudinal bracing welded into plate. With the completion of the two pile bents braced together, stability was achieved, and the barge was removed as lateral support. Construction of the remainder of the platform continued in both directions outward of the central bents.

The sequence of erection of the two intermediate mooring dolphins proceeded in similar manner.

The end mooring dolphins consisted of four pile clusters of three batter piles per cluster. Each cluster was driven as a unit and lashed together, after which the clusters were connected to obtain stability. Low water bracing again was employed to support the scaffolding for capping of the pile groups.

Submarine Pipeline

Prior to the construction of the wharf, a 22-inch coated submarine pipeline was laid to connect the shore facilities to the island berth. The pipe was supplied with a 5/8-inch Somastic coal-tar enamel coating for protection against corrosion and a 1-7/8-inch Heavycote concrete coating to protect the enamel coating against abrasion during and after placing and to overcome buoyancy of the line. The installation of the line involved hazards not commonly met in such an operation because of the strong tidal currents and the steep "drop-off" on the bottom profile located about 500 feet offshore. The line was assembled to full length on the shore in a shallow trench which was flooded after the line was completed and tested. The floating pipeline, 1,700 feet long, was towed into position by means of a winch-mounted barge firmly anchored in position and by a tug.

A pulling sled at the outshore end of the line consisted of a 14-foot length of 20-inch piping equipped with a blind flange and lug fastened to a 10-foot by 6-foot steel plate sled which was streamlined to minimize drag during pulling. The sled also acts as a spread footing to restrict settlement of the pipeline into the river bottom. The shoreward end of the pipeline was laid in a gradually sloping trench to overcome the sharp "drop-off" occasioned by an underwater ledge. A minimum of 18 inches of cover was provided along the shore end to protect the pipe from breakers and scour.

The submarine line is connected to a 20-inch riser located slightly off center of the breasting platform. The riser pipe is supported by a pipe collar bearing on the wharf deck and is braced by several underwater struts to prevent the riser from vibrating in the current. The underwater bracing was designed for installation without the use of divers. The pipe riser is electrically isolated from the wharf proper by means of micarta insulation. The underwater connection of the riser to the submarine line was simplified by using lap joint flanges at both ends of the riser pipe, which coupled with the flexibility at the free end of the riser made alignment of the pipe sections relatively simple.

Protection Against Corrosion

Water temperature, salinity, and high humidity in the atmosphere all contribute to a high rate of corrosion of steel in Sumatra. Various means were considered by the engineers to protect the steel members from corrosion. These included the use of vinyl paints, bituminous coatings, gunite (sprayed concrete), metalizing, Monel metal sheathing, special alloy steels (rust inhibiting), and cathodic protection. Each of the three zones of varying corrosion required its own special treatment dictated by economy, accessibility, life expectancy of the protection to be furnished, ease of maintenance, and skill required for installation. As a result of these studies and the experience gained during the construction and maintenance of Wharf No. 1, the following protective procedures were established:

(1) For the atmospheric zone, exposed metal was coated with pigmented, rust-inhibitive paints, while steel decking, cap beams and the like were given a two-coat application of Bitumastic 50 with a finish coat of Bituplastic 28 to prevent "alligatoring".
(2) In the splash zone from Elevation +2 to about Elevation +13, the pipe piles were coated with three coats of hot applied Bitumastic Enamel 70B, reinforced with two layers of Fiberglas matting. A vinyl paint system consisting of five coats of Nukemite was used on the fender panels because of the difficulty in applying the reinforced, hot enamel to WF sections.
(3) The submerged steel is not provided with any protective coating but a cathodic protection system is provided to counteract any corrosion of the steel submerged in water and mud.

The cathodic protection for the wharf consists of an impressed voltage system using Duriron anodes suspended beneath the wharf structure. The water at the site consists of ocean water diluted by the fresh waters from the Siak River and has a pH value of 8.00 and a chloride content of 15,930 parts per million. A cathodic protection current of five milliamperes per square

foot of pile surface submerged in water has been provided, which offers a margin of safety over the three milliamps per square foot which is frequently used for protecting structures in sea water. This safety factor takes into consideration any inequalities in current distribution caused by proximity and interference effects from the somewhat restricted anode locations.

The use of an impressed voltage form of cathodic protection was selected over a sacrificial anode type system which requires frequent replacement and consequent high maintenance costs. Various types of anode materials were considered, including scrap iron, graphite and Duriron. Although Duriron costs more than the other materials, it possesses definite advantages over graphite with regard to electrical connection detail and electrical characteristics.

Remote anode beds as well as anodes within the confines of the berth were considered. The nature of the bottom, which consists of a thin sand stratum overlying a thick layer of clay, is such that the anodes lying on the bottom would sink into the clay with deleterious effects. These effects include accelerated attack on the anodes plus possible gas blocking and increased anode voltage drop as the anodes become embedded. Suspended anodes beneath the wharf structure overcome these difficulties and were, therefore, used. Due to the ebb and flow tidal currents, the suspended anodes, if hung freely from the wharf, would have excessive movement. The wake of ships' propellers would also violently disturb the anodes. The anodes were, therefore, secured both top and bottom by means of Saran rope to prevent movement and accidental contact with the steel structure. Saran, a plastic material, was selected because of its strength and resistance to salt water action and marine borers.

The Somastic coated submarine pipeline is equipped with insulating flanges at both the wharf end and the shore end of the pipe. At the present time, however, the insulating flanges at the wharf end are shunted to permit protection of the submarine line by the wharf cathodic protection system. As time passes and the mastic coating on the pipeline deteriorates, the electric load impressed on the wharf system will increase. If this load becomes excessive, an additional rectifier will be provided at the shore end and connected to a future anode bed also located at the shore end to provide a separate cathodic protection system for the submarine line.

Fender System

To withstand the large impact forces that occur during docking and undocking of vessels, the wharf is equipped with a resilient fender system designed to decelerate a vessel's approach gradually and avoid injury to the vessel and wharf. The fenders are designed to accommodate a 50,000 d.w.t. tanker berthing with a velocity component of 16 feet per minute normal to the wharf at the instant of impact. Four 40-ton steel springs are required for each fender panel, each spring assembly consisting of three concentrically-mounted, helical, steel spring coils which can be compressed a maximum of 13 inches.

Fender panels are located only on the breasting platform and the intermediate mooring dolphins since they are the only structures which will be in direct contact with the vessel during berthing. The panels are constructed of steel H-piles interconnected by steel wales and struts. All steel surfaces that may be in contact with vessels are faced with hardwood rubbing strips.

Marine borers flourish in Sumatran waters and atmospheric rot is accelerated by the high humidity. Hardwoods were, therefore, required to resist borer attack and to provide resistance to decay and mechanical abrasion. Since local hardwoods were not available, timbers were obtained in Singapore. Balau, a dense Malayan timber, was selected for the fender strips because it could be used to resist marine borers without creosoting. Kempas, a tough local hardwood, was picked for the decking but, to improve its useful life, a preservative treatment was used. Since pressure treating facilities were not available locally, Kempas had to be purchased in Singapore, where it was impregnated with Tanalith. White oak from the United States, used on Wharf No. 1, suffered severely from borer attack below mean tide level, requiring replacement within several years of its installation. Untreated Douglas fir, used in the same installation for deck planking, also was short-lived because of its susceptibility to atmospheric rot.

Wharf Equipment

The wharf is equipped with two stiff-legged derricks for hose handling. Each derrick has two 40-foot booms with a capacity of 5,000 lbs. per boom. The arrangement at each berth for loading tankers consists of a manifold with 12-inch flanges for attachment of two 12-inch rubber hoses. For catchment of oil spillage and dripping, a slop system supported under the deck has been provided.

Contracts and Cost of Construction

The wharf structures were constructed by J. C. Gammon (Malaya) Ltd. The submarine pipeline was assembled and laid by Caltex engineers and local labor employed in the oil fields.

The total cost of the project was approximately $1,700,000.

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Plate 2. — View of Loading Platform.

PILE DETAILS AT LOADING PLATFORM

PLATE
3

Plate 2. — View of Loading Platform.

PILE CLUSTER
AT END MOORING DOLPHIN

PLATE
4

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WATER FORCES ON ACCELERATED CYLINDERS

A. D. K. Laird,[1] C. A. Johnson,[2] and R. W. Walker[3]

ABSTRACT

For single horizontal cylinders the drag coefficients during acceleration agreed with uniform motion values, but deviations occurred during deceleration. The acceleration modulus did not correlate the resistance coefficients near boundary layer transition. A leading parallel cylinder reduced the mean resistance but caused some violent force fluctuations.

INTRODUCTION

The economical design of fixed structures immersed in ocean waves or other oscillating fluid masses is handicapped by an insufficiency of data. Statistical studies based on wave records collected in many parts of the world provide an estimate of the heights and steepnesses of waves which may be expected at a given site during a period of years. From these wave parameters and the depth of the water, the designer must calculate the forces on the structures.

Measurements of the forces of waves on structures of many shapes and configurations have been made on models in the laboratory and in the ocean. A semi-empirical method has been developed at the University of California(1) for the prediction of forces induced by waves in moderately deep water. Data from vertical circular cylinders have been reported(2) recently by the Wave Research Laboratory of the University of California. These data spread about the corresponding semi-empirical predictions in a manner which suggests

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1. Prof. of Mechanical Eng., Univ. of Calif., Berkeley, Calif.
2. Research Asst., Inst. of Eng. Research, Univ. of Calif., Berkeley, Calif.
3. Research Asst., Univ. of Calif., Berkeley, Calif. Presently: Design Engr., Caterpillar Tractor Co., Peoria, Ill.

that some important factors are missing from this method of prediction. It was also typical of other published data that, in many cases, the forces exerted by the majority of the waves could be predicted by the method developed at the University of California; but, in a significant number of cases, the predicted forces were up to three times too large or too small. These numerous failures of the analytical method based on the currently recognized phenomena indicated the existence of undiscovered phenomena in the interaction of waves and piling.

At the University of California ocean test site, water depth was of the order of one-tenth of the wave length. The records of the shapes of the waves showed deviations from those predicted by the corresponding wave theories. These deviations could be attributed to irregularities of the bottom, to the superposition of small waves on larger waves, to the neighboring piling and to imperfections of the theory. In the majority of cases, the force records agreed within 30 per cent with the predictions from the theory. Many of the small deviations synchronized with the deviations of the wave shapes, indicating small effects caused by small deviations of the actual from the theoretical. Many of the waves were short crested rather than long crested as specified by the theory, but no consequent large effects were detected. The large deviations did not appear to result from any of the observed discrepancies. Previous laboratory studies at the University of California had shown that the pile spacing was large enough so that the shapes and sizes of the waves were not appreciably affected. The effect of the pile spacing on the forces on the piles was not investigated. In the semi-empirical method, it was assumed that the magnitude, direction and rate of change of acceleration do not affect the drag coefficient. No data were available to check these assumptions.

Research was initiated by Laird to test the validity of the semi-empirical method under controlled conditions and to discover which of the variables that had previously been neglected were actually important. The part of this research which dealt with the effects of accelerations produced by essentially constant driving forces has been summarized by Keim.(3) He accelerated half-inch and one-inch horizontal cylinders vertically through water from their rest positions. His work showed no large deviations up to accelerations of 4 ft/sec^2 and velocities of 1.5 ft/sec. The effects of spacing and of other types of accelerations upon circular cylinders remained to be investigated.

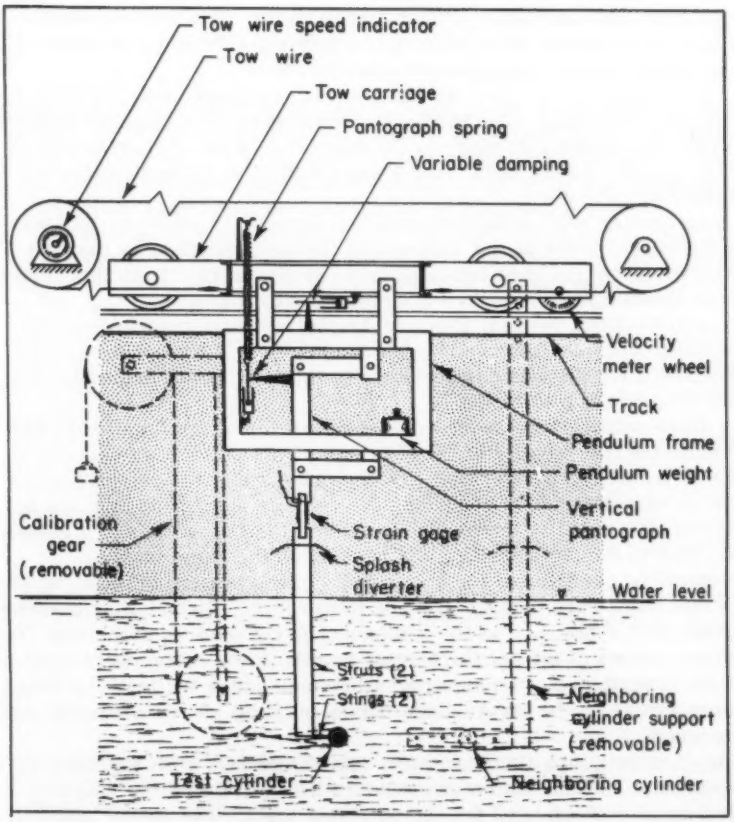
The present paper deals with the forces exerted by water on an immersed, horizontally mounted, circular cylinder, which was moved in a horizontal direction normal to its long axis. The accelerations and velocities were in the range common to ocean waves. Both constant and variable linear accelerations and decelerations were used to discover possible effects of acceleration and rates of change of acceleration. Three diameters were used to cover the range of Reynolds numbers from the Karman vortex region through the boundary layer transition region. The regions of overlapping of the data permitted observation of the effects of diameter at constant Reynolds number. By towing another similar cylinder parallel to the test cylinder, some of the effects of a neighboring cylinder were investigated.

Experimental Equipment

The experiments were performed in the 200 ft by 8 ft towing tank at the Engineering Field Station of the University of California.

The cylinders were of smooth aluminum with hemispherical ends. The surface condition was held constant by coating the cylinders with white petroleum jelly. Their overall length was 39 in. The diameters were 1.25, 2.33 and 4.47 in., which resulted in length to diameter ratios of 31.2, 16.7 and 8.72. The apparatus used to test the smallest cylinder was described previously.(4) In Fig. 1 the apparatus for the tests on the larger cylinders is shown diagrammatically. For the larger cylinders, the water depth was 48 in. The cylinder top to water surface distance was 13 in.

The cylinders were supported by 1/4 in. diameter horizontal stings in the wake of the cylinders just inboard from their hemispherical ends. The vertical struts of 0.25 x 1.5 in. steel were about 2.5 in. behind the cylinders. The



EXPERIMENTAL APPARATUS

FIGURE 1

upper ends of the struts were thickened to accommodate an SR-4 strain gauge bridge which served as a force meter. The output of the bridge was recorded by an Offner Dynograph at a tape speed of 50 mm per sec.

The struts were attached to a pantograph and its load, consisting of the test cylinder and its supports, was borne by two vertical soft coil springs attached to the tow carriage frame. Adjustable damping was provided by a dashpot. The pantograph was attached to a horizontal pendulum which was hung from the tow carriage by four links pivoted on ball bearings. Adjustable damping was obtained by a dashpot with a valved bypass. Weights placed on the pendulum frame reduced deflections caused by the cylinder drag. The pendulum effectively reduced transmission of disturbances from the wire to the cylinder. The pantograph isolated the cylinders from vertical vibrations of the tow carriage during preliminary test. It was found that these vibrations were negligible and since the vertical forces on the cylinders caused objectionable vibrations of the pantograph, it was inactivated by clamping it to the horizontal pendulum during test runs.

When a neighboring cylinder was to be used, a support was bolted to the front of the tow carriage. The position of the neighboring cylinder was varied by selecting convenient bolt holes in the support. These supports were at the ends of the neighboring cylinder which was nearly the same diameter as the test cylinder and about 9 in. longer.

The force meter was calibrated at one to three hour intervals by applying a known horizontal force at the midpoint of the test cylinder, by means of a wire. The wire was led over two pulleys as shown in Fig. 1, so that weights could be suspended from a hook at the other end of the wire.

The controls of the General Electric Amplidyne were modified so that constant linear accelerations and decelerations up to 0.3 g could be maintained for arbitrary periods compatible with the tank length and with chosen maximum velocities up to 22 fps.

The voltage output of the speed control generator of the Amplidyne was a linear function of the velocity of the carriage. This voltage was recorded continuously on the Dynograph tape. During most runs, the velocity trace was calibrated by simultaneously recording the velocity during the uniform part of the motion on a velocity meter which counted the revolutions of a calibrated wheel attached to the two carriage and running on the same track.

The natural frequencies of the components of the system in their possible modes of vibration were measured. The modes were separated by means of a 2g accelerometer fastened to the component in the desired direction. The simultaneous traces of the accelerometer and the force meter were used to estimate the magnitudes of transmitted vibrations under the test conditions.

The drag forces on the struts alone during uniform and accelerated motions were measured.

The effect of length to diameter ratio was checked by a preliminary set of experiments with 2.33 in. diameter cylinders over the complete range of speeds and accelerations used for the principal tests. The ratios used were 9.02, 12.9 and 16.7.

Method of Analysis

The forces of the water on the test cylinder during a run can be represented[5] in terms of a resistance coefficient, C, defined by Eq. (1)

$$D_T = CS\,\rho\,\frac{V^2}{2} \quad (1)$$

where D_T is the total drag, S is the area of a diametral cross section of the cylinder, ρ is the fluid density, and V is the velocity of the cylinder, relative to the water. Many of the available resistance coefficient data have been summarized by Crooke[6] who correlated them with the acceleration modulus, AD/V^2.

According to the semi-empirical method of reference (1), the resistance can be separated into a drag force which is due to velocity effects, and an inertia force caused by the acceleration of the surrounding water. The drag coefficient, C_D, is defined by Eq. (2), and the inertia coefficient, C_M, is given by Eq. (3).

$$\text{Drag force} = C_D\,S\rho\,\frac{V^2}{2} \quad (2)$$

$$\text{Inertia force} = C_M\,M\,.\,A \quad (3)$$

where A is the acceleration and M the mass of the water displaced by the cylinder. This method of analysis implied that the drag and the inertia effects are without appreciable mutual interference, so that they are additive. It also requires a knowledge of the variation of C_D and C_M with the flow conditions. This method has been amplified by Keulegan.[5]

In the analysis of the present paper, because the variation of C_M is practically unknown, it was assumed constant and equal to unity, as predicted by simple hydrodynamic theory. This simplification has the effect of including in C_D the deviations of C_M from unity.

The velocity and force meter records were transferred to rectangular coordinates as shown in Fig. 4. The existence of constant acceleration was indicated by a straight portion of the velocity curve. The slope of the velocity curve at any time was equal to the acceleration.

The drag and inertia forces on the struts and stings were subtracted from all the force records. It was assumed that the struts experienced the same drag at a given velocity during accelerated motion as during uniform motion. The change in drag on the struts caused by the presence of the wake of the cylinder was assumed negligible. The inertia force of the water acting on the struts was neglected.

The coefficients for all cylinders were corrected to infinite length to diameter ratio, L/D, by means of the factors given by Wieselberger and Betz.[7]

Results

The principal test conditions are listed in Table 1, the data during acceleration in Table 2, and during uniform motion in Table 3.

The infinite cylinder drag coefficients, C_D, are shown as a function of Reynolds number in Figs. 2a, b and c, for the test cylinders alone. The data of the two larger cylinders pass through the transition from laminar to turbulent boundary layers. It should be noted that although the drag coefficient correction from finite to infinite length cylinders is not valid beyond the transition velocity, it was used in this paper for want of a better correction.

The insensitivity of the drag coefficients to changes in acceleration is an important result. The larger the cylinder, the greater was the tendency for

TABLE 1 RANGE OF VARIABLES

CYLINDER WITHOUT NEIGHBOR

Diameter, in.	1.25	2.33	4.47
Length to diameter ratio	31.2	16.7	8.72
Reynolds Number, $x10^{-4}$	2.5 to 82	2.7 to 31	2.7 to 41
Acceleration, ft/sec^2	-23 to 10	-5 to 6	-4 to 5

CYLINDER WITH ONE PARALLEL NEIGHBOR LEADING

Diameter in in. of					
Test Cylinder	2.33			4.47	
Neighboring Cylinder	2.37			4.50	
Reynolds Number $x10^{-4}$	1.7 to 21			3.3 to 28	
Acceleration, ft/sec^2	1.5 to 3.6			1.2 to 2.3	
Centerline Distance, in.					
Vertical	0	1.38	2	0	2.63
Horizontal	6.0	8.1	5.7	9.0	15.0
	9.0	9.1		15.0	15.0
	11.0				
	13.0				

the drag coefficients during deceleration to deviate from those during acceleration. This may be due to diameter or to length to diameter ratio effects.

Another interesting result of this research is the demonstration in Fig. 3 that the acceleration modulus, AD/V^2, where A is the acceleration, D the projected diameter and V the velocity, does not correlate resistance coefficients, C, in the region of small values of the modulus. It is particularly impotent near the boundary layer transition.

In Fig. 4 examples of typical velocity and force traces are represented for three runs during acceleration, uniform motion and deceleration. These curves, from which small fluctuations have been faired out, show typical variations of force as functions of time and velocity in the present research. They may be used as rough models for corresponding ocean waves. The higher set of velocity and force curves for the 2.33 in. cylinder (Run 125) shows clearly the effect of boundary layer transition. Because of boundary layer transition on the faster 4.47 in. cylinder (Run 108), its force trace is lower than that of the lower of the 2.33 in. cylinders (Run 218). In each curve the inertia effects are evident in the increased force during acceleration over the force during deceleration at the same velocity. Typical force fluctuations which were faired out of the uniform velocity sections of Runs 218 and 108 are shown enlarged in Figs. 5a and 5b respectively.

The typical changes in the drag force traces when a neighboring cylinder is introduced are shown in Figs. 5a and 5b. In Fig. 5a, the reduction of the mean force, or sheltering, by the neighboring cylinder can be seen. Differences in spacing of the cylinders cause differences in mean force and in fluctuating force. In Fig. 5b, the sheltering and the fluctuations are seen to be more pronounced for the 4.47 in. than the 2.33 in. cylinder. Additional data are included in Table 3.

TABLE 2 CONSTANT ACCELERATION DATA SUMMARY

NO Neighbor. Corrected to Infinite Length

2.33 in. CYLINDER

Accel. ft/sec^2	Accel. Mod.	Resist. Coef.	Drag Coef.	Run No.
2.0 fps Vel.. 0.347 x 10^F N_R				
-2.50	-0.121	1.20	1.45	152R
-2.56	-0.124	1.30	1.60	152
-2.92	-0.142	1.57	1.87	147
3.0 fps Vel., 0.520x10^5 N_R				
5.59	0.121	1.32	1.04	119
5.32	0.115	1.47	1.2.	121
4.71	0.102	1.34	1.12	118
4.69	0.101	1.46	1.25	112
3.76	0.0810	1.50	1.34	123
3.70	0.0798	1.42	1.25	116
2.91	0.0626	1.44	1.30	155
2.78	0.0599	1.33	1.20	216
2.71	0.0584	1.37	1.25	109
2.13	0.0458	1.29	1.19	145
2.10	0.0454	1.28	1.19	147
2.08	0.0448	1.33	1.24	153
2.03	0.0439	1.32	1.23	218
2.03	0.0437	1.31	1.22	152R
2.02	0.0436	1.37	1.28	154
2.00	0.0437	1.37	1.29	152
1.97	0.0425	1.35	1.26	154R
-2.56	-0.0555	1.38	1.50	152
-2.92	-0.0632	1.31	1.44	147
5.0 fps Vel., 0.866 x 10^5 N_R				
5.82	0.0451	1.54	1.44	126
5.53	0.0431	1.37	1.28	128
5.32	0.0412	1.38	1.29	121
4.71	0.0365	1.35	1.27	118
4.69	0.0363	1.34	1.27	112
4.34	0.0336	1.66	1.32	117
3.88	0.0302	1.39	1.33	124
3.79	0.0294	1.30	1.23	110
3.76	0.0291	1.41	1.35	123

Accel. ft/sec^2	Accel. Mod.	Resist. Coef.	Drag Coef.	Run No.
5.0 fps Vel., o.866 x 10^5 NR (Cont'd)				
3.70	0.0287	1.33	1.27	116
3.51	0.0273	1.33	1.27	155R
3.41	0.0265	1.23	1.18	155
3.31	0.0256	1.23	1.18	216
2.79	0.0216	1.60	1.56	144
2.71	0.021	1.26	1.21	109
2.13	0.0165	1.25	1.22	145
2.11	0.0164	1.35	1.31	146
2.08	0.0161	1.18	1.15	153
2.03	0.0158	1.42	1.38	218
2.02	0.0157	1.17	1.21	154
1.97	0.0153	1.29	1.25	154R
7.0 fps Vel., 1.21x10^5 N_R				
6.09	0.0241	1.34	1.29	127
5.82	0.0230	1.40	1.35	126
5.59	0.0222	1.24	1.19	119
5.53	0.0220	1.28	1.23	128
5.32	0.0211	1.28	1.24	121
4.71	0.0186	1.34	1.30	118
4.39	0.0174	1.22	1.18	124
4.34	0.0172	1.33	1.29	117
3.76	0.0149	1.39	1.36	123
3.70	0.0147	1.29	1.26	116
3.51	0.0139	1.26	1.23	155R
3.41	0.0135	1.21	1.18	155
3.31	0.0131	1.24	1.22	216
1.97	0.0078	1.20	1.18	154R
9.0 fps Vel., 1.56 x 10^5 N_R				
6.09	0.0146	1.25	1.22	127
5.53	0.0132	1.22	1.19	128
4.39	0.0105	1.17	1.15	124
4.34	0.0104	1.27	1.25	117
3.76	0.0090	1.28	1.26	123

In Fig. 6, effects of a neighboring cylinder during constant acceleration are shown for the 4.47 in. cylinder. These curves for the 4.47 in. cylinder were obtained by subtracting the calculated forces, at the corresponding acceleration and velocities for the cylinder alone, from the measured forces during acceleration. It is evident that increased acceleration caused earlier onset of the sheltering effect. A corresponding increase in the magnitude of the sheltering and in the violence of the fluctuating forces occurred. During acceleration, the violence of the fluctuation was greater for the offset cylinders than for the cylinders in line. Similar results were obtained with the 2.33 in. cylinders, but in general the fluctuations were less severe.

Accel. ft/sec^2	Accel. Mod.	Resist. Coef.	Drag Coef.	Run No.
9.0 fps Vel., 1.56 x 10^5 N_R (contd)				
3.51	0.0084	1.15	1.13	155R
4.47 in. CYLINDER				
0.8 fps Vel., 0.266 x 10^5 N_R				
-1.26	0.732	1.41	3.17	49
1.0 fps Vel., 0.333 x 10^5 N_R				
-1.26	-0.468	1.66	2.79	49
2.0 fps Vel., 0.666 x 10^5 N_R				
-1.44	-0.134	1.18	1.51	66
3.0 fps Vel., 0.998 x 10^5 N_R				
4.32	0.179	1.42	0.99	120
4.25	0.176	1.74	1.31	122
3.67	0.152	1.51	1.15	111
3.35	0.139	1.61	1.28	121
3.29	0.136	1.59	1.26	102
3.21	0.133	1.52	1.20	110
3.15	0.131	1.69	1.37	126
3.04	0.126	1.68	1.38	112
2.74	0.113	1.45	1.17	117
2.39	0.0987	1.55	1.31	125
2.32	0.0962	1.56	1.33	108
2.29	0.0950	1.52	1.29	116
2.11	0.0875	1.57	1.36	60
2.08	0.0859	1.45	1.24	91
1.83	0.0759	1.43	1.24	107
1.72	0.0713	1.39	1.22	52
1.65	0.0684	1.48	1.32	86
1.24	0.0515	1.39	1.27	50
-1.44	-0.0535	1.35	1.50	66

Accel. ft/sec^2	Accel. Mod.	Resist. Coef.	Drag Coef.	Run No.
5.0 fps Vel., 1.66 x 10^5 N_R				
4.60	0.0860	1.17	1.00	111
4.25	0.0634	1.22	1.07	122
4.18	0.0622	1.19	1.03	112
4.16	0.0620	1.11	0.96	110
3.86	0.0576	1.30	1.17	121
3.29	0.0490	1.30	1.18	102
3.23	0.0481	1.14	1.03	109
3.15	0.0470	1.16	1.03	126
2.74	0.0408	1.06	0.96	117
2.45	0.0365	1.17	1.08	116
2.30	0.0342	1.20	1.07	125
2.26	0.0337	1.13	1.05	107
1.57	0.0234	1.13	1.07	98

Discussion

The infinite length drag coefficients for the three sizes of cylinders in Fig. 2 during acceleration and uniform motion agree with the accepted values for Reynolds numbers below transition. The grouping of the constant acceleration points results from calculating at a few arbitrarily chosen velocities for the sake of economy and clarity of presentation. Had more velocities been chosen, more similar groups would have resulted. The frequencies of the occurrence of points are shown by the numbers beside them, unity having been omitted. Reliable data at velocities below 3 fps were difficult to obtain at reasonable accelerations because of starting disturbances such as shown in Fig. 4 for Run 125 during the first half second. In Figs. 2a, b and c, the scatter is larger at the lower velocities because of the relatively larger inaccuracies in calibration, measurement and calculation. During deceleration

TABLE 3 UNIFORM MOTION DATA SUMMARY

Vel.	N_R	Measured Drag, lb.			Drag Coefficient			Run	Period, Seconds		
fps	$\times 10.^{-5}$	Mean	Max	Min.	Mean	Max.	Min.		Measured Major	Minor	Eddy Pairs
					2.33 in. NO NEIGHBOR						
1.59	0.276	1.68	2.25	1.10	1.47	1.97	0.96	2	0.068		0.65
1.87	0.324	2.04	2.75	1.35	1.29	1.74	0.85	148	0.186		0.55
1.87	0.324	1.98	2.75	1.20	1.25	1.74	0.76	148R	0.186	0.093	0.55
1.87	0.324	2.04	2.75	1.31	1.29	1.74	0.83	149	0.100		0.55
1.87	0.324	2.28	3.06	1.51	1.44	1.94	0.96	149R	0.150		0.55
1.87	0.324	2.19	2.75	1.67	1.39	1.74	1.06	150	0.170	0.065	0.55
1.87	0.324	2.28	2.75	1.82	1.44	1.74	1.15	151	0.170	0.053	0.55
1.98	0.343	2.61	3.37	1.85	1.47	1.90	1.04	3	0.059		0.53
2.26	0.392	2.98	4.12	1.84	1.30	1.78	0.80	8	0.160	0.078	0.46
2.65	0.459	4.15	5.22	3.13	1.31	1.64	0.99	13	0.062		0.39
2.67	0.463	3.55	4.84	2.26	1.10	1.50	0.70	12	0.153		0.39
3.18	0.551	6.58	7.45	5.75	1.44	1.63	1.26	17	0.160		0.33
3.26	0.565	6.38	7.82	4.97	1.33	1.63	1.03	19	0.152		0.32
4.05	0.702	10.1	11.8	8.48	1.36	1.59	1.14	21	0.160	0.080	0.26
6.38	1.11	22.4	25.8	19.1	1.22	1.40	1.04	220	0.062	0.055	0.15
6.42	1.11	22.4	23.9	21.1	1.20	1.28	1.13	153R		0.050	0.15
6.44	1.12	22.5	24.7	20.6	1.20	1.32	1.10	147	0.080	0.060	0.15
6.44	1.12	22.9	24.7	21.6	1.22	1.32	1.15	152	0.060		0.15
6.45	1.12	22.2	23.8	20.6	1.18	1.27	1.10	152R	0.070	0.152	0.15
7.20	1.25	26.8	29.6	24.0	1.14	1.26	1.02	146	0.155	0.062	0.13
7.21	1.25	26.1	27.7	24.5	1.11	1.18	1.04	153	0.060		0.13
9.05	1.57	45.0	49.8	40.3	1.21	1.34	1.09	218	0.062		0.11
9.19	1.59	41.8	44.2	39.6	1.09	1.16	1.04	154	0.060		0.11
9.19	1.59	41.8	44.2	39.6	1.09	1.16	1.04	154R	0.060		0.11
9.32	1.62	45.0	47.6	42.5	1.14	1.21	1.08	110	0.062		0.10
9.32	1.62	46.0	51.2	41.2	1.17	1.30	1.05	145	0.062		0.10
9.41	1.63	46.1	50.1	42.4	1.15	1.25	1.06	109	0.060		0.10
9.49	1.64	46.9	49.5	44.1	1.15	1.21	1.08	111	0.062		0.10
10.57	1.83	57.0	60.5	53.6	1.13	1.20	1.06	108			0.092
10.66	1.85	57.0	60.5	53.6	1.11	1.18	1.04	107	0.062		0.091
11.98	2.08	71.1	76.3	66.3	1.09	1.17	1.02	216	0.082		0.081
11.99	2.08	70.5	73.6	67.4	1.09	1.13	1.04	155	0.085		0.081
12.09	2.09	71.8	76.5	67.3	1.09	1.16	1.02	144	0.070	0.062	0.080
12.11	2.10	69.5	73.4	65.6	1.05	1.11	0.99	155R	0.060		0.080
12.82	2.22	73.7	85.1	62.4	0.99	1.14	0.84	121	0.060		0.076
12.82	2.22	73.2	81.5	64.8	0.98	1.10	0.87	122	0.060		0.076
12.92	2.24	73.7	81.6	66.0	0.98	1.08	0.87	118	0.060		0.075
12.92	2.24	75.6	83.9	67.2	1.00	1.11	0.89	120	0.060		0.075

there appeared to be a diameter, or length to diameter ratio, effect which caused the drag coefficients to be higher for larger diameters at the lower velocities. If the coefficient of mass were taken as zero, then the coefficient of resistance would be the coefficient of drag. In Run 107 the deceleration points in Fig. 2c below a Reynolds number of 10^5 correspond to points in Fig. 3 for the same run at values of AD/V^2 less than -0.13. It can be seen that these resistance coefficients are much closer to the uniform motion and acceleration drag coefficients than are the deceleration drag coefficients. This fact suggests that the mass coefficient may be closer to zero than to unity during deceleration in this range of Reynolds numbers. The deceleration

Vel. fps	N_R $\times 10^{-5}$	Measured Drag, lb. Mean	Max.	Min.	Drag Coefficient Mean	Max	Min.	Run	Period, Seconds Measured Major	Minor	Eddy Pairs
12.97	2.24	74.8	83.9	65.9	0.99	1.11	0.87	115	0.072		0.075
13.04	2.26	74.7	82.6	67.1	0.97	1.07	0.87	119	0.062		0.074
17.21	2.98	49.1	54.3	43.9	0.37	0.41	0.33	124	0.060		0.056
17.21	2.98	48.0	54.2	42.2	0.36	0.44	0.32	129	0.055		0.056
17.30	3.00	51.2	59.1	43.7	0.38	0.44	0.32	126	0.059		0.056
17.38	3.01	52.2	55.6	48.7	0.38	0.41	0.36	128	0.065		0.056
17.40	3.02	51.2	54.0	48.8	0.37	0.39	0.36	125	0.062		0.056
17.40	3.02	49.2	55.1	45.6	0.36	0.40	0.33	132	0.060		0.056
17.40	3.02	47.5	54.2	40.9	0.35	0.40	0.30	133	0.060		0.056
17.43	3.02	48.7	50.5	47.0	0.35	0.37	0.34	130	0.060		0.056
17.55	3.04	53.9	66.0	41.9	0.39	0.47	0.30	123	0.060		0.055
17.57	3.04	50.6	56.8	44.8	0.36	0.41	0.32	127	0.060		0.055
18.07	3.13	53.8	61.6	46.2	0.36	0.42	0.31	131	0.059		0.054
					2.33 in. WITH NEIGHBOR 6.0 in. AHEAD IN LINE						
1.87	0.324	-0.2	0.6	-1.0	-0.12	0.37	-0.62	162	0.176	0.072	0.55
1.87	0.324	-0.5	0.4	-1.4	-0.32	0.26	-0.91	161	0.170	0.060	0.55
6.38	1.11	4.7	12.5	-3.1	0.26	0.68	-0.17	156	0.088		0.15
7.09	1.23	7.4	16.4	-1.5	0.33	0.72	-0.06	157	0.084		0.14
8.72	1.51	8.5	21.3	-4.0	0.25	0.62	-0.12	158	0.095		0.11
8.72	1.51	13.5	24.1	2.9	0.39	0.70	0.08	159	0.100	0.080	0.11
11.50	1.91	23.4	37.3	9.4	0.39	0.62	0.16	160	0.072		0.084
					2.33 in. WITH NEIGHBOR 9.0 in. AHEAD IN LINE						
1.87	0.324	0.6	1.6	-0.4	0.38	0.98	0.23	174	0.090		0.55
1.87	0.324	-0.0	0.8	-0.8	-0.03	0.48	-0.53	174R	0.172	0.091	0.55
1.87	0.324	0.6	1.7	-0.4	0.38	1.09	-0.28	175	0.060		0.55
6.37	1.10	5.7	8.9	2.6	0.31	0.48	0.14	170R	0.086	0.120	0.15
6.38	1.11	7.9	13.7	2.1	0.43	0.75	0.11	170	0.172		0.15
7.11	1.23	6.4	11.5	1.4	0.28	0.50	0.06	171R	0.145	0.090	0.14
7.13	1.24	8.1	15.4	0.9	0.35	0.67	0.04	171	0.088		0.14
8.82	1.53	15.2	26.8	3.5	0.43	0.76	0.10	172R	0.088		0.11
8.90	1.54	16.1	32.4	-0.1	0.45	0.90	-0.00	172	0.088	0.200	0.11
11.52	2.00	16.3	26.5	6.2	0.27	0.44	0.10	173R	0.060		0.084
11.62	2.02	31.8	53.5	5.6	0.52	0.88	0.09	$173R_2$	0.066		0.083
11.81	2.05	30.5	48.7	12.3	0.48	0.77	0.19	173	0.062	0.070	0.082

points in Fig. 2b at Reynolds numbers less than 10^5 correspond approximately to points in Fig. 3 at AD/V^2 values between -0.02 and -0.09. It appears that more research would be necessary to explain the high values of drag coefficient below a Reynolds number of about 10^5, and the low values above about 1.8×10^5 during deceleration.

The boundary layer transition in Figs. 2b and 2c appears early for infinitely long smooth cylinders. This probably reflects the finite length of the cylinder, the flexibility of the supports and the disturbances in the water.

During some of the fast runs of the 4.47 in. cylinder, a surface wave up to 4 in. high followed the cylinder about 10 in. behind. It was estimated approximately (8) that such a wave would cause a maximum overestimate of the resistance coefficient of less than 3 per cent. Since this was less than the general scatter it was neglected.

TABLE 3 (continued)

Vel fps	N_R x10.$^{-5}$	Measured Drag lb. Mean	Max.	Min.	Drag Coefficient Mean	Max.	Min.	Run	Period, Seconds Measured Major	Eddy Minor	Pairs
2.33 in. WITH NEIGHBOR 11.0 in. AHEAD IN LINE											
1.87	0.324	0.9	1.9	-0.0	0.58	1.19	-0.03	180	0.168	0.065	0.55
1.87	0.324	0.7	1.6	-0.2	0.42	0.98	-0.13	181	0.11		0.55
6.32	1.09	7.1	13.6	0.5	0.39	0.75	0.03	176	0.15	0.087	0.15
6.32	1.09	8.6	17.6	-0.3	0.48	0.98	-0.02	176R	0.089		0.15
7.08	1.23	8.3	17.8	-1.1	0.37	0.79	-0.05	177	0.089	0.150	0.14
8.63	1.50	15.8	22.6	9.0	0.47	0.67	0.27	178	0.060		0.11
11.59	2.00	27.3	43.3	11.4	0.45	0.71	0.19	179	0.060		0.084
2.33 in. WITH NEIGHBOR 13.0 in. AHEAD IN LINE											
1.87	0.324	0.9	1.9	-0.0	0.58	1.19	-0.03	187			0.55
1.87	0.324	0.9	1.9	-0.0	0.58	1.19	-0.03	188			0.55
6.32	1.09	8.5	12.3	2.9	0.47	0.68	0.16	183	0.090		0.15
6.33	1.10	8.4	13.3	3.6	0.46	0.73	0.20	183R	0.088		0.15
6.99	1.21	9.9	17.4	2.4	0.45	0.79	0.11	184	0.086		0.14
8.58	1.49	14.4	20.5	8.4	0.43	0.62	0.25	185	0.123	0.068	0.11
11.52	2.00	26.6	37.6	15.9	0.44	0.63	0.27	186	0.058		0.084
2.33 in. WITH NEIGHBOR 5.7 in. AHEAD 2 in. OFFSET UP											
1.87	0.324	1.6	2.0	1.1	0.98	1.29	0.68	197	0.090	0.165	0.55
1.87	0.324	1.3	1.9	0.8	0.82	1.19	0.48	198	0.090		0.55
6.33	1.10	3.8	8.5	-0.8	0.21	0.47	-0.04	193	0.083		0.15
6.99	1.21	16.4	30.5	2.4	0.74	1.38	0.11	194	0.086		0.14
8.39	1.45	21.7	35.3	8.1	0.68	1.11	0.26	195	0.082		0.12
11.81	2.05	13.4	23.7	3.1	0.21	0.38	0.05	196	0.082		0.082
2.33 in. WITH NEIGHBOR 8.1 in. AHEAD 1.4 in. OFFSET UP											
1.87	0.324	1.2	2.2	-0.1	0.78	1.39	-0.07	199	0.088		0.55
1.87	0.324	1.0	1.7	-0.1	0.62	1.09	-0.07	200	0.170		0.55
1.87	0.324	0.8	1.7	-0.0	0.52	1.09	-0.03	201	0.087		0.55
6.25	1.08	8.0	19.2	-3.2	0.45	1.08	-0.20	209	0.087	0.15	0.15
6.30	1.09	9.2	23.9	-5.5	0.51	1.33	-0.31	202	0.087		0.15
6.30	1.09	8.7	27.8	-10.3	0.49	1.55	-0.57	202R	0.088		0.15
6.30	1.09	10.8	31.1	-9.3	0.60	1.73	-0.52	202R$_2$	0.088		0.15
6.91	1.20	9.9	30.4	-10.5	0.46	1.41	-0.49	208	0.086		0.14
6.94	1.20	11.4	27.5	-4.8	0.52	1.26	-0.22	203	0.085		0.14
8.02	1.39	13.3	28.0	-1.4	0.46	0.96	-0.05	204	0.088		0.12

The velocity meter was separated from the cylinder by the towing wire which produced the negligible maximum error in carriage position of only 3 hundredths of an inch.

After each run, long period waves were damped out very slowly so that it was impractical to wait for the waves to stop before the next run. Consequently, runs were started in the 2 or 3 seconds during which the water was practically motionless in the nodes of the waves. There was some chance that the start of a run would not be at a node. The nodes did not last during the whole of most runs, so that these long waves could cause errors in velocity up to 1 fps for fast runs and large cylinders. There is a possibility that the large drag coefficients during deceleration are caused by these wave induced velocities. It is improbable, however, that such errors would always be towards

TABLE 3 (continued)

Vel	N_R	Measured Drag lb.			Drag Coefficient			Run	Period, Seconds		
fps	x_{10}^{-5}	Mean	Max.	Min.	Mean	Max.	Min.		Measured Major	Eddy Minor	Pairs
2.33 in. WITH NEIGHBOR 8.1 in. AHEAD 1.4 in. OFFSET UP											
8.29	1.44	15.1	36.5	-6.3	0.49	1.17	-0.20	207	0.082		0.12
10.83	1.88	32.5	50.2	14.8	0.61	0.95	0.28	206	0.057	0.082	0.089
11.25	1.95	29.0	46.8	11.2	0.51	0.82	0.20	205	0.086		0.086
2.33 in. WITH NEIGHBOR 9.1 in AHEAD 1.4 in OFFSET UP											
1.86	0.324	0.8	1.9	-0.4	0.48	1.20	-0.23	210	0.59	0.10	0.55
1.87	0.324	0.7	1.7	-0.4	0.42	1.09	-0.23	211	0.58	0.087	0.55
6.23	1.08	5.9	18.2	-6.6	0.33	1.04	-0.38	212R	0.090	0.185	0.15
6.32	1.09	4.0	16.8	-8.9	0.22	0.93	-0.49	212	0.18	0.080	0.15
6.98	1.21	10.9	22.7	-1.0	0.49	1.03	-0.05	213	0.090	0.16	0.14
8.39	1.45	14.1	26.0	2.2	0.44	0.82	0.07	214	0.076	0.164	0.12
11.06	1.92	23.5	41.3	5.8	0.43	0.75	0.11	215	0.057		0.088
4.47 in. NO NEIGHBOR											
1.55	0.516	2.2	2.9	1.4	1.19	1.59	0.79	1	0.18	0.73	1.30
1.55	0.516	1.9	2.6	1.2	1.06	1.46	0.66	2	0.18	0.080	1.30
1.55	0.516	2.6	3.6	1.6	1.45	2.01	0.88	5	0.18	0.080	1.30
1.65	0.549	2.8	3.5	2.1	1.37	1.70	1.05	9	0.18	0.089	1.22
1.65	0.549	2.6	3.7	1.4	1.26	1.83	0.69	12	0.18		1.22
1.65	0.549	2.7	3.4	2.0	1.34	1.69	0.98	15	0.18	0.075	1.22
1.86	0.619	3.5	4.9	2.1	1.35	1.88	0.82	20	0.18	0.065	1.08
1.87	0.622	3.2	3.9	2.6	1.22	1.47	0.98	19	0.185	0.063	1.08
1.87	0.622	3.7	4.9	2.6	1.42	1.86	0.98	22	0.18	0.065	1.08
1.96	0.652	3.9	5.3	2.4	1.34	1.85	0.84	27	0.18	0.06	1.02
2.23	0.742	5.3	6.3	4.3	1.38	1.65	1.12	36	0.18	0.062	0.90
2.25	0.749	5.1	6.3	3.9	1.33	1.66	1.02	38	0.18	0.063	0.89
2.64	0.878	6.8	8.3	5.4	1.30	1.58	1.03	43	0.064	0.18	0.76
2.64	0.878	6.6	7.9	5.3	1.26	1.51	1.01	45	0.62	0.20	0.76
3.18	1.06	9.9	11.5	8.2	1.30	1.52	1.08	50	0.63	0.175	0.58
3.18	1.06	10.4	12.1	8.7	1.37	1.59	1.39	53	0.18	0.063	0.58
4.02	1.34	13.6	15.7	11.5	1.12	1.29	0.95	61	0.186		0.46
4.02	1.34	14.8	16.9	12.7	1.22	1.39	1.05	63	0.185	0.063	0.46
5.35	1.78	22.0	23.9	19.3	1.02	1.11	0.90	66	0.18	0.065	0.35
5.35	1.78	20.0	21.7	18.4	0.93	1.01	0.85	69	0.182	0.07	0.35
5.35	1.78	20.5	32.6	18.4	0.95	1.52	0.85	77	0.18	0.10	0.35

higher values as shown in Figs. 2b and 2c. There were undoubtedly other disturbances in the tank, such as large eddies, which could have contributed to the general scatter of the data.

As a test of the applicability of the Wieselberger and Betz[7] correction factors for conversion of data from finite to infinite cylinders, the drag coefficients for the three lengths of 2.33 in. diameter cylinders were corrected to infinite lengths by their factors. All three sets fell about the accepted values below the transition Reynolds number. Through transition and above, however, more research is needed to establish correction factors.

TABLE 3 (continued)

Vel	N_R	Measured Drag lb.			Drag Coefficient			Run	Period, Seconds		
fps	$\times 10.^{-5}$	Mean	Max.	Min.	Mean	Max.	Min.		Measured Eddy Pairs Major	Minor	
4.47 in. NO NEIGHBOR											
6.30	2.10	23.8	26.8	20.9	0.80	0.90	0.70	79	0.18	0.067	0.30
6.30	2.10	24.0	26.2	21.8	0.80	0.88	0.73	82	0.18	0.06	0.30
6.32	2.10	23.3	25.9	20.9	0.78	0.86	0.70	83	0.18	0.065	0.29
6.36	2.12	23.3	25.9	20.8	0.77	0.85	0.69	88	0.17	0.067	0.29
7.46	2.48	24.8	27.3	22.3	0.59	0.65	0.53	91	0.175		0.25
7.48	2.49	24.8	30.3	19.2	0.59	0.72	0.46	93	0.173		0.25
7.48	2.49	23.7	28.3	19.1	0.56	0.67	0.45	96	0.172		0.25
8.36	2.78	29.3	32.5	26.1	0.56	0.62	0.50	99	0.160		0.22
8.26	2.75	29.8	35.3	24.2	0.58	0.69	0.47	106	0.183		0.23
8.36	2.78	29.8	34.4	25.2	0.57	0.66	0.48	101	0.187		0.22
9.35	3.11	39.5	46.5	34.6	0.60	0.71	0.53	110	0.20	0.174	0.20
9.55	3.18	39.1	42.9	35.3	0.58	0.63	0.52	114	0.195	0.17	0.19
10.21	3.40	46.2	52.4	40.1	0.59	0.67	0.51	118	0.175	0.08	0.18
10.25	3.41	43.1	49.3	37.1	0.55	0.63	0.47	119	0.184	0.08	0.18
12.37	4.11	61.1	69.7	53.7	0.53	0.61	0.46	126	0.185		0.15
12.37	4.11	61.1	64.4	55.9	0.53	0.56	0.48	127	0.075		0.15
4.47 in. WITH NEIGHBOR 9.0 in. AHEAD IN LINE											
3.29	1.10	1.2	3.7	-0.5	0.15	0.45	-0.06	137	0.21		0.57
4.27	1.42	3.5	7.4	-0.3	0.26	0.54	-0.02	136	0.245		0.44
4.96	1.65	6.2	16.8	-4.4	0.34	0.91	-0.24	135	0.235		0.37
5.39	1.79	4.9	14.5	-4.7	0.23	0.66	-0.22	134	0.204		0.35
5.91	1.97	4.6	17.9	-6.5	0.16	0.63	-0.23	133	0.208		0.31
6.28	2.09	3.5	8.3	-1.3	0.12	0.28	-0.04	128	0.10	0.20	0.30
7.51	2.50	9.9	65.6	-45.7	0.23	1.55	-1.08	130	0.18		0.25
8.25	2.75	6.5	91.0	-78.1	0.13	1.78	-.153	132	0.18		0.23
4.47 in. WITH NEIGHBOR 15.0 in. AHEAD IN LINE											
3.26	1.09	3.0	5.3	0.8	0.38	0.66	0.10	151	0.27		0.57
3.90	1.30	6.2	9.3	3.0	0.54	0.82	0.26	152	0.18		0.48
5.28	1.76	16.1	23.8	8.4	0.77	1.14	0.41	153	0.17		0.35
6.60	2.20	20.3	42.3	-1.9	0.62	1.29	-0.06	154	0.17		0.28
7.74	2.58	20.1	46.4	-6.1	0.45	1.03	-0.14	155	0.16		0.24

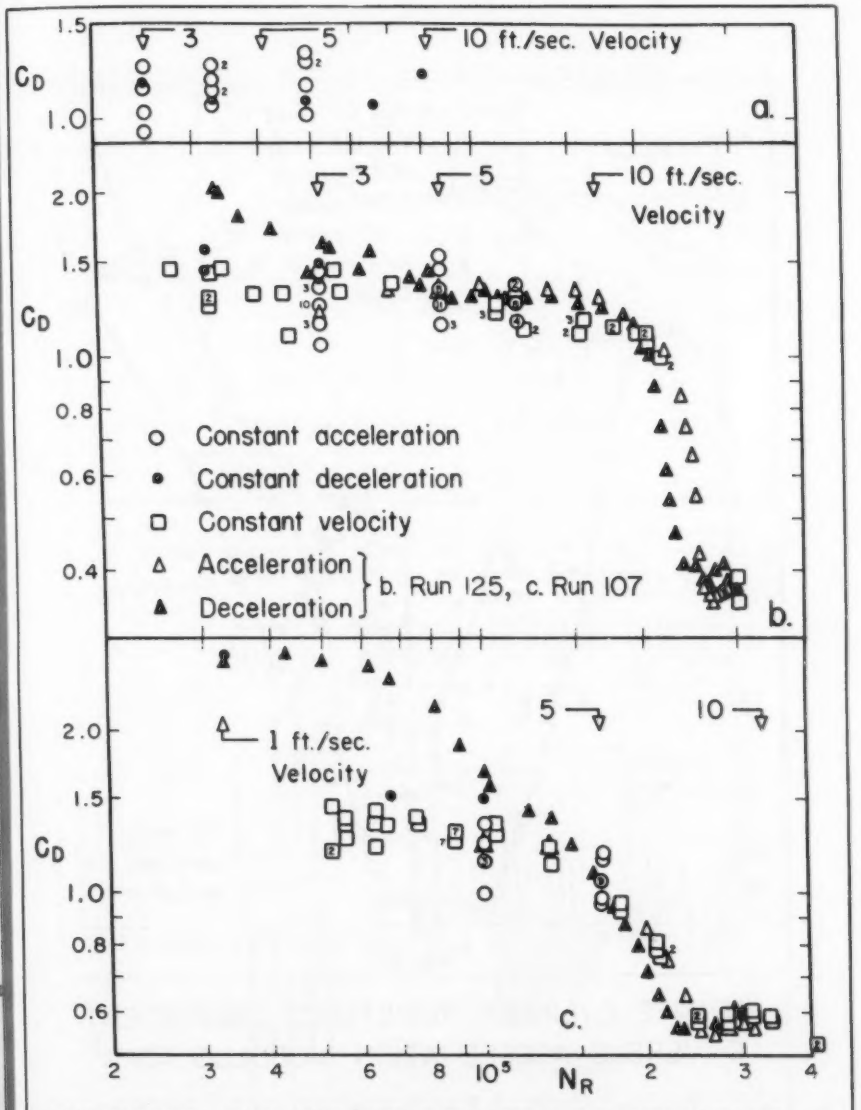
The use of unity for the inertia coefficient, C_M, was indicated by the simplicity of the calculations and because its actual values were practically unknown. If the acceleration and the drag coefficient during part of a run were constant, the mass coefficient could be calculated from the force-time curve. This would be an attractive method if precise data were available. Many values of C_M were calculated from selected curves, but different pairs of selected points on the same curve gave different values of C_M. Most of the values were below unity and tended to decrease and become negative at increasing values of time, because C_D was not constant but decreased as the Reynolds number approached the transition value. It is probable that the

TABLE 3 (continued)

Vel	N_R	Measured Drag lb.			Drag Coefficient			Run	Period, Seconds	
fps	$\times 10.^{-5}$	Mean	Max.	Min.	Mean	Max.	Min.		Measured Eddy Pair Major	Minor
4.47 in. WITH NEIGHBOR 9.0 in. AHEAD 2.6 in. OFFSET UP										
5.18	1.72	10.7	50.8	-29.5	0.53	2.52	-1.46	141	0.168	0.36
6.38	2.12	10.7	42.4	-21.0	0.35	1.39	-0.69	142	0.16	0.29
8.00	2.66	5.0	46.4	-36.5	0.10	0.97	-0.76	143	0.10	0.23
4.47 in. WITH NEIGHBOR 15.0 in. AHEAD 2.6 in OFFSET UP										
3.24	1.07	3.7	5.6	1.8	0.47	0.71	0.22	144	0.16	0.57
5.24	1.74	15.1	21.8	8.4	0.73	1.06	0.42	145		0.35
5.25	1.75	14.6	22.5	4.6	0.71	1.09	0.22	146	0.22	0.35
6.58	2.19	20.3	35.6	4.9	0.62	1.10	0.15	148	0.22	0.28
6.66	2.22	16.4	27.9	5.5	0.49	0.84	0.17	147	0.175	0.28
7.85	2.61	18.8	46.2	-8.5	0.41	1.00	-0.18	150	0.165	0.24
7.91	2.67	16.2	46.4	-14.1	0.34	0.99	-0.30	149	0.165	0.24

actual values of C_M were close to unity. It is apparent that deviations of C_M from unity had little effect on the acceleration and uniform motion results over the range of variables of the present research, and the analysis of Eqs. (2) and (3) is adequate for these cases.

The variations of the resistance coefficient, C, as a function of the acceleration modulus, AD/V^2, is shown in Fig. 3. The coordinates were distorted for the sake of clarity. The data from the three sizes of cylinders scatter widely about the proposed line given by Crooke,[6] at these low values of the modulus. For all non-zero values of acceleration, the boundary layer must pass through transition twice as the acceleration modulus goes from large positive values through zero to large negative values. The magnitude of the acceleration modulus at which transition will occur depends mostly on the diameter, acceleration, and velocity, but, also on the roughness and fluid disturbances. By proper choice of variables, therefore, transition could be induced at almost any value of the modulus. Since the resistance coefficient reflects the change in drag coefficient, the correlation of C by the acceleration modulus is not possible where transition occurs. A continuous set of values for each of the 2.33 and the 4.47 inch cylinders from near rest through positive, zero, and negative accelerations to near rest is shown for Runs 125 and 107 respectively. The value of C for uniform velocity (acceleration modulus zero) for the larger cylinder is higher because of the overcorrection for the length to diameter ratio at speeds beyond the transition speed. Since both runs started and ended with relatively large accelerations, the resistance coefficients have very large positive values near starting and very large negative values near stopping. The scatter is large when the velocities are 1 fps or less. Consequently, the curves are shown dotted to indicate trends rather than exact values. Below transition, the uniform motion value of resistance coefficient is approximately 1.25 to 1.30. Since the acceleration modulus does not correlate data in the Reynolds number range of the present work, and



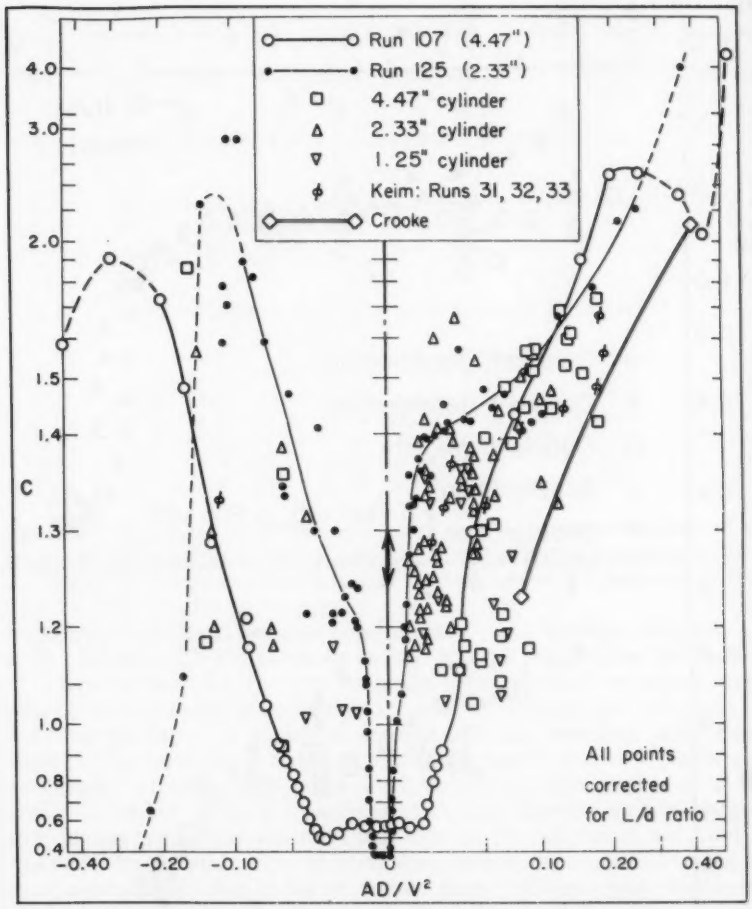
INFINITE CYLINDER DRAG COEFFICIENT C_D VERSUS REYNOLDS NUMBER N_R

a. d = 1.25 in., L = 39 in., L/d corr. factor = 1.26

b. d = 2.33 in., L = 39 in., L/d corr. factor = 1.36

c. d = 4.77 in., L = 39 in., L/d corr. factor = 1.53

FIGURE 2

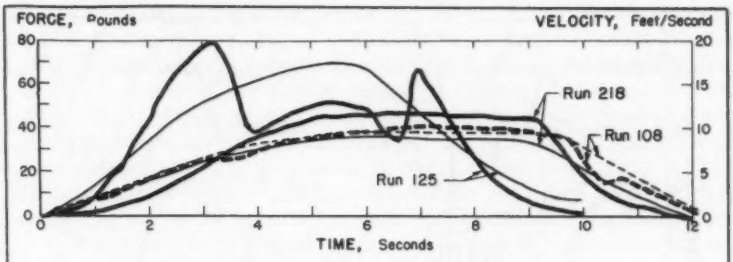


INFINITE CYLINDER RESISTANCE COEFFICIENT C VERSUS ACCELERATION MODULUS AD/V^2

—·— Zero value for uniform motion

<—> Approximate uniform motion value below transition

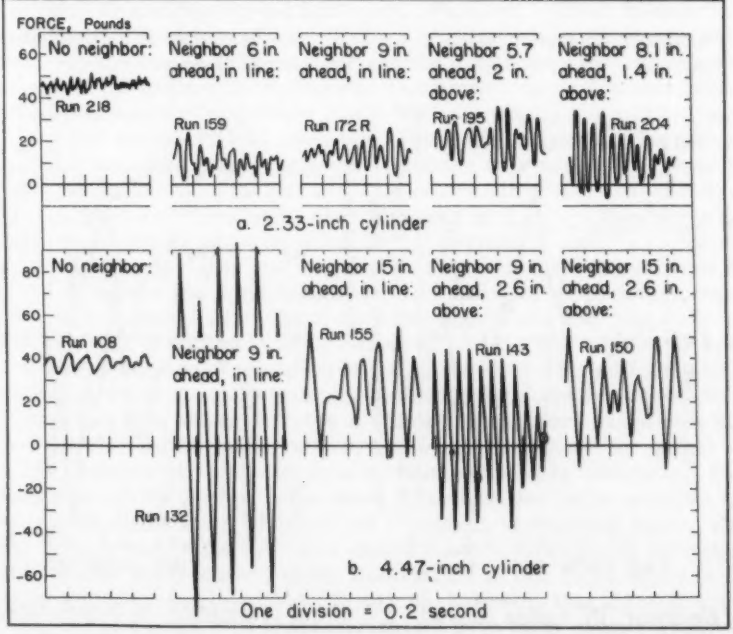
FIGURE 3



TYPICAL FORCES AND VELOCITIES
AS FUNCTIONS OF TIME

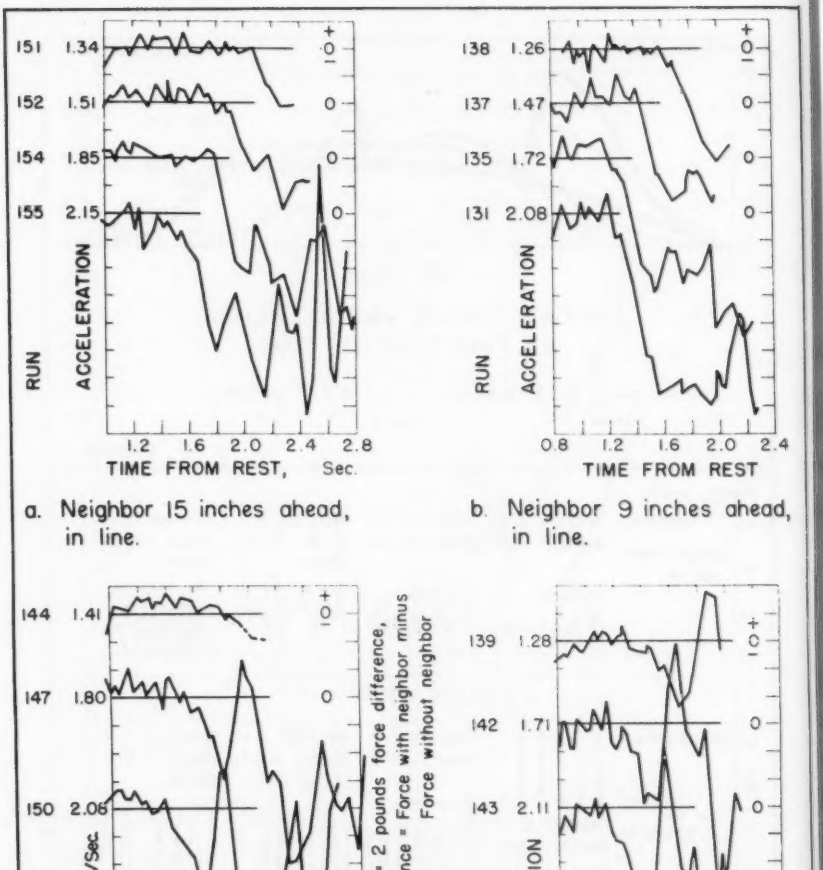
Velocity: —— 2.33" cylinder, - - - 4.47" cylinder
Force: ▬▬ " " ▬ ▬ " "

FIGURE 4



UNIFORM MOTION DRAG FORCES FOR CYLINDER ALONE
AND WITH NEIGHBOR PLACED AS STATED

FIGURE 5



a. Neighbor 15 inches ahead, in line.

b. Neighbor 9 inches ahead, in line.

c. Neighbor 15 inches ahead, offset 2.6 inches above.

d. Neighbor 9 inches ahead, offset 2.6 inches above.

EFFECT OF NEIGHBORING CYLINDER ON FORCE EXERTED BY 4.47-INCH DIAMETER TEST CYLINDER

FIGURE 6

since they were taken from runs at different accelerations, speeds and diameters, the remainder of the points in Fig. 3 appear to be unrelated.

In Fig. 4, Runs 108 and 125 have about the same maximum Reynolds numbers, 3.18 and 3.02 x 10^5 respectively. Run 108 was practically the same as Run 107. Consequently, Fig. 4 shows the force-time-velocity relationships, Fig. 3 the resistance coefficient-acceleration modulus variations and Figs. 2b and 2c the drag coefficient-Reynolds number variations for Runs 107-8 and 125. It can be seen that the maximum Reynolds number does not completely specify the shape of the curves. The effects of transition and acceleration are particularly noticeable in Fig. 4. Run 218 for the 2.33 inch cylinder was a little slower than Run 108 for the 4.47 inch cylinder, but the forces were higher over most of the run because Run 218 was below transition. In Figs. 5a and 5b typical force traces are reproduced to the same scale as in Fig. 4. In most runs, the fluctuations were absent for the single cylinders during rapid acceleration and deceleration. The fluctuations appeared to be due to the interaction of the forces caused by the nearly periodic shedding of eddies during uniform motion, and the elastic forces of the strain bars. Similar behavior of smokestacks has been reported by Penzien,[9] and of iron bars by Landweber.[10] By means of the accelerometer, it was demonstrated that no measurable horizontal vibration was transmitted from the tow-wire or carriage to the horizontal pendulum. Direct measurement of vertical vibrations of the cylinder was not attempted because it was submerged. On the basis of the cylinder and the displaced water being a weight on the end of the quarter inch stings as cantilevers, the calculated natural period for the vertical vibrations was 0.065 seconds for the 2.33 in., and 0.092 seconds for the 4.47 in. cylinder. The force meter did not respond to transverse horizontal forces, and was ten times as sensitive to forces in the direction of motion as in the vertical transverse direction. The torsional vibrations about a vertical axis, and vibrations in the direction of motion of the 2.33 inch cylinder had measured natural periods of about 0.13 seconds. These torsional vibrations and vibrations in the direction of motion of the 4.47 inch cylinders had periods of about 0.15 and 0.18 respectively, when the cylinder was stopped. After rapid decelerations of the 4.47 inch cylinder, the vibrations in the direction of motion frequently had a period of 0.16 secs. The measured periods of vibration of the cylinders during uniform motion are shown in Table 3. For comparison, the approximate periods of shedding eddy pairs are also shown in Table 3. These values were based on a Strouhal number of 0.186 for Reynolds numbers below 10^5 and 0.200 for higher Reynolds numbers at which the Strouhal number is more uncertain. From a consideration of the flow patterns about cylinders, one might expect a transverse, or lift, force from eddies at the frequency of shedding pairs. Each change in direction of the lift force should cause a force in the direction of motion, so that the frequency of the fluctuations in this force would be that of shedding alternate single eddies. It may be, however, that if transverse vibrations at some preferred frequency should have large enough amplitudes that eddy pairs would be shed at this frequency. If the vibration were in the direction of motion, one might expect a tendency toward shedding either single alternate eddies or simultaneous pairs. An examination of periods in Table 3 shows practically no agreement with the periods estimated from Strouhal numbers at either full or half periods. The periods for the single 2.33 in. cylinder are seen to be predominantly near 0.06 sec, which probably corresponds to the vertical vibration. The less frequent and generally smaller

vibrations between 0.15 and 0.17 secs may be due to the vibrations in the direction of motion. For the single 4.47 in. cylinder, the principal period, 0.18, is that of the natural vibration in the direction of motion. The period of approximately 0.08 secs may correspond to the natural period of the vertical vibrations of the frame of the car. During Run 126 of the 4.47 in. cylinder, the period was 0.075 and after some 3 seconds, the stings broke due to fatigue. Consequently, it would appear that vertical vibrations of the stings had occurred frequently throughout the runs. During Run 132 the frame of the carriage appeared to vibrate with an amplitude of over 1/2 in. At the same time, the carriage wheels were periodically lifted clear of the track due to the forces from the test cylinder and its neighbor. The total weight of the carriage and its load was estimated at 450 lbs. Apparently the lift forces were comparable to the drag forces. Further research to measure the lift forces and to determine the eddy patterns and the reasons for the observed periods would be desirable.

The increase of resistance force with time during constant acceleration is given in Fig. 4 for cylinders without neighbors. In Fig. 6, the first portion of each curve that oscillates about zero represents similar behavior, but with somewhat more vibration. The sudden decrease of values shows the onset of the influence of a parallel neighbor leading the test cylinder. It is believed that the arrival of a shed eddy, or eddies, at the test cylinder changed the flow about it and reduced the mean force. The subsequent arrival of more eddies caused changes in the flow pattern that resulted in large fluctuations in the resistance force. Evidently, eddies from offset neighboring cylinders affected the following cylinder differently than did eddies from cylinders directly ahead. These traces might be likened to what would be expected during the corresponding motion of water during the crest half of the wave action. There is an essential difference in the two systems, however, since the water in the wave would have been disturbed by eddies shed by the pair into the water during the latter part of the trough half of the preceding wave. During deceleration of the towed cylinder there was no return of the resistance force to that of a cylinder alone. That is, the sheltering continued throughout deceleration.

CONCLUSIONS

From the test conditions of this research the following conclusions may be drawn:

1. Drag coefficients for accelerated and uniform motion agreed with the corresponding accepted values for uniform motion.
2. During deceleration there was strong evidence of deviation of the drag coefficient from the accepted values for uniform motion.
3. The acceleration modulus, A^D/V^2, did not correlate the resistance coefficient near boundary layer transition.
4. Some irregularities in wave force records may have been due to transition and acceleration effects.
5. The presence of a neighboring cylinder caused reductions of resistance and large fluctuations in the resistance and lift forces. The spacing was important in these effects.

ACKNOWLEDGMENT

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Proceedings of the American Society of Civil Engineers

"TIDAL" NAVIGATIONAL PROBLEMS BELOW WILSON DAM[a]

Rex A. Elder,[1] M. ASCE

SUMMARY

As part of the present modernization program of the navigational facilities in the immediate vicinity of the Tennessee Valley Authority's Wilson Project, the present 14,500-foot canal is to be realigned so as to provide a more adequate approach to the downstream canal entrance, and the canal will be deepened 10 feet. The deepening will eliminate a 10-foot lift lock which is now located just upstream from the mouth of the canal. The present and realigned canals have a swing-span railway bridge located at their entrances. The changes in the canal design, plus contemplated additions in the turbine settings at the Wilson powerhouse with a consequent increase in maximum probable turbine discharge from 70,000 to 100,000 cfs, necessitated a study of the navigational problems that might occur at the canal entrance with the proposed facilities.

A fixed-bed 1:100 undistorted-scale model was chosen as the most expeditious method of studying the proposed design. The tests evinced two specific flow conditions which could cause considerable difficulty to tows attempting to enter the canal. Steady discharges in the neighborhood of full-turbine capacity cause reasonably high-velocity flow across the entrance and make passage through the railway swing span bridge pier openings and into the canal extremely difficult. This problem is made even more acute by the daily and hourly fluctuations in the flows originating from the turbines at the dam. Present and contemplated future turbine loading patterns are expected to result in what can be equated to a twice daily "tide." The turbines will be shut off during the night and will then come to nearly full discharge during the early morning hours. In midday the flows will be considerably reduced, will rise to near maximum discharges in the early evening, and then will be almost completely shut down by about midnight. With a contemplated 100,000 cubic feet per second turbine capacity, the river stage in the vicinity of the canal entrance can be expected to fluctuate up to 5.6 feet during a period as

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a. Presented at the October, 1958 ASCE Convention in New York, N. Y.

1. Head, Hydr. Operations and Tests Section, Hydr. Data Branch, TVA, Norris, Tenn.

short as 1-1/2 hours. The model tests showed that this "tide" would create a most acute problem in the immediate vicinity of the railway bridge, since with a maximum incoming "tide" a severe cross current occurred under the bridge whereas with an outgoing or falling "tide" the current in the canal entrance was parallel to the canal. Thus a pilot could expect anything from a paralleling velocity to one nearly at right angles to his direction of motion.

A modification in the entrance shape was developed which effectively eliminated the two cross-current problems. This modification consisted of the extension and broadening of the downstream point of land between the canal and the river and the placing of six 32-foot diameter sheet-piling cells on 57-foot centers parallel to the river and downstream from the railway bridge. The effect of these changes was to shield the entrance from the steady-flow cross currents and to break up and realign the "tidal" currents so that the right passage through the railway bridge swing span contained no appreciable cross current under any flow condition.

Historical Background

Historically, navigation of the Tennessee River dates from 1540 when De Soto used it on his trip west to the Mississippi River. Naturally, the Indians used the river long before De Soto and long afterwards. Except for a few short years, history records that the major obstacles to navigation on the Tennessee River have been located in the northwest corner of Alabama near the city of Florence. Originally, the obstacles consisted primarily of the dangerous Muscle and Elk River Shoals. For almost 100 years man attempted to alleviate the problems associated with these shoals. In 1836 a 14.5-mile

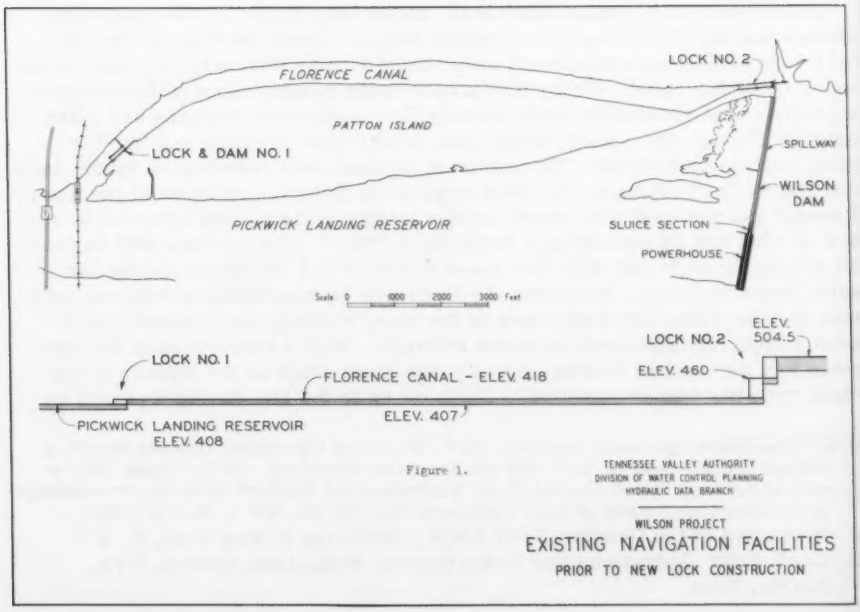


Figure 1.

canal, incorporating 17 locks and skirting the Muscle Shoals, was opened. The primitive state of navigation engineering, lack of maintenance funds, and railroad competition made this venture unsound from the start, and it was abandoned within a year. Between 1875 and 1890 the U. S. Army Engineers built two canals totaling just over 18 miles in length and incorporating 11 locks which bypassed the Muscle and Elk River Shoals. This canal provided a 5-foot depth and was used successfully until 1926 when the Wilson Project was put into operation by the Corps of Engineers, U. S. Army. The Wilson Project was started during the First World War, but was not finished until 1926. The facilities at this project are shown in Fig. 1. They consist of a 90-foot high concrete gravity dam, a double-lift lock located on the right bank at the north end of the dam, a 2-3/4-mile canal named the Florence Canal along the right side of Patton Island, and Lock and Dam No. 1 with a 10-foot height near the downstream end of the Florence Canal. The lock sizes vary slightly, the smallest being 60 feet wide x 292 feet long. The dam contains a 58-gate spillway, a sluice section, and an 18-turbine powerhouse.

At completion the Wilson facilities were capable of providing a 9-foot channel from Lock and Dam No. 1 to the head of Wilson Reservoir, but downstream from Wilson only a 4-foot channel existed, while upstream a 2-foot channel was available to Chattanooga and a 1-1/2-foot channel from there to Knoxville. These shallow channel depths effectively blocked any major use of the river.

In 1933 the Wilson facilities, except for operation of the lock, were transferred to the newly created Tennessee Valley Authority and since then have been incorporated into the 9-foot channel facilities which TVA put into service as far as Chattanooga in April 1945 and on to Knoxville by November 1952. The completion of each phase of the main-river navigational program has resulted in a marked increase in river traffic. This increase was so marked that by 1955 the Wilson facilities were again becoming the major obstacle to

Figure 2.--24-BARGE TOW ON TENNESSEE RIVER

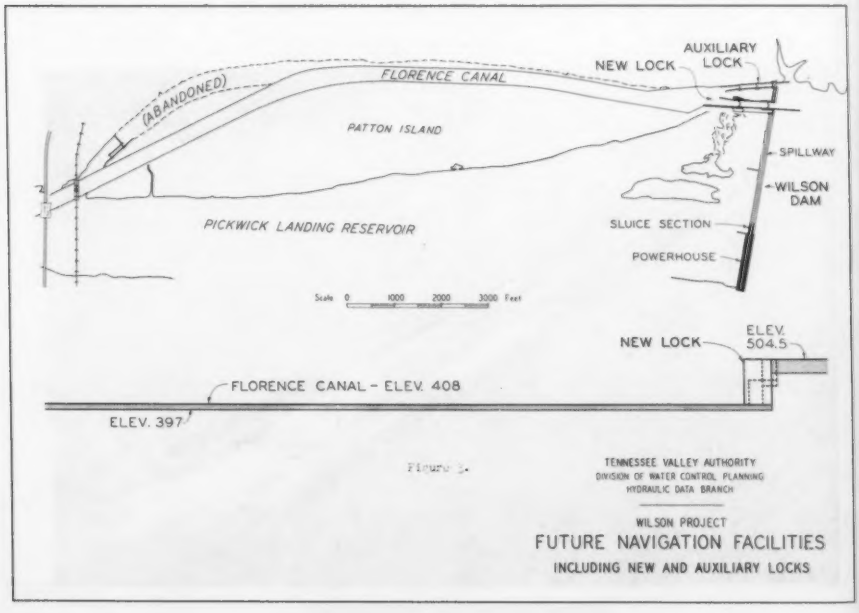
navigation on the Tennessee River. Tows similar in nature to the one shown in Fig. 2 were starting to use the river. Such a tow required up to 24 hours to pass through the three lock chambers.

To alleviate this bottleneck, TVA in 1955 was granted an appropriation to start design of a new lock and canal facilities and in 1956 construction was started. Initial operation of the new structures will be made in 1959 with project completion in 1960.

The new facilities are shown in Figs. 3 and 4. These consist of a single 100-foot lift, 110 feet wide x 600 feet long lock and a realigned and deepened canal. The canal will be deepened by approximately 10 feet to allow direct passage from Pickwick Reservoir to the new lock. The canal realignment occurs primarily in the downstream third and was made to provide a better approach through the railway bridge swing span section. Old Lock and Dam No. 1 will be left in place because cost of removal is not justified. The new facilities will pass tows of the size shown in Fig. 2 in two lockages, or in a maximum time of 60 minutes. The lower chamber of the old double-lift lock at the dam will be rebuilt by deepening the chamber 10 feet so that this lock can be used for auxiliary lock purposes.

Canal Entrance Problems

Navigation into and out of the present Florence Canal has been difficult under even nominally good river conditions because of the presence of the railway bridge at the mouth of the canal. This should be quite apparent from an examination of Fig. 1. With even moderate flows, conditions were made worse due to the current cutting angularly across the navigation channel.



With the construction of the realigned canal, two entrance problems were envisaged. The first was the cross-current problem experienced with the present canal, while the second was a new cross-current problem created by the opening up of the 2-3/4-mile canal.

The river stage at the mouth of the canal is determined by two factors: the rate of discharge at Wilson Dam, and the water surface level in Pickwick Reservoir. Since Pickwick Reservoir rises and falls very slowly, the water surface in the canal can be expected to adjust to any Pickwich changes without any apparent effect. This will not be true of the stage changes brought about by discharge changes at Wilson Dam. The Wilson hydro plant at present has a total discharge capacity of 70,000 cfs, and an additional 30,000 cfs is proposed for the near future. In the combined steam and hydro generated power system operated by TVA it has been found best to operate the hydro plants for peaking purposes. In such a situation, experience has shown that Wilson will follow a loading pattern which during most of the time can be somewhat generalized as follows: The plant will be essentially shut down from midnight to 5 or 6 a.m.; the load will then be picked up rapidly and in from 1-1/2 to 2 hours full capacity may be in use; the load will then fall off to perhaps half capacity until 4 to 5 p.m. when a second increase will start and by 8 p.m. full capacity may be required. After 10 p.m., the load will drop off fairly rapidly until by midnight it will be essentially zero. In the next few years it is anticipated that this loading pattern will become even more pronounced. It was envisioned that the effect of such discharge variations on the flows at the entrance to the canal could be quite appreciable. Every sudden or rapid stage change at the canal entrance would require a subsequent readjustment of the stage in the canal in essentially the same manner as tidal flows occur in river estuaries. Since the canal is 2-3/4 miles long and averages over 370 feet wide, very appreciable quantities of water must flow into or out of the canal to bring about any stage readjustment. It was contemplated that the

Figure 4.--AERIAL VIEW OF FUTURE NAVIGATION FACILITIES

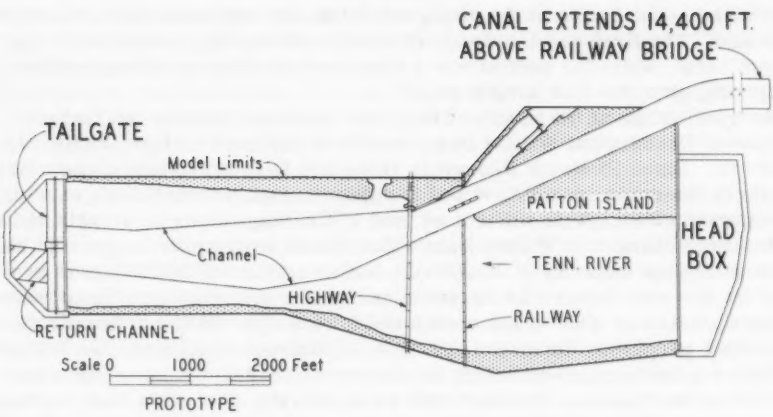


Figure 5.--FLORENCE CANAL ENTRANCE MODEL

velocity and direction of these "tidal" flows would create serious navigational problems in the vicinity of the railway bridge.

Model Studies

To ascertain the magnitude of the contemplated canal entrance problems and to develop a solution to any such delineated problems, a 1:100 undistorted-scale, fixed-bed model of the canal and the river in the vicinity of the canal entrance was constructed at TVA's Hydraulic Laboratory. The basic features

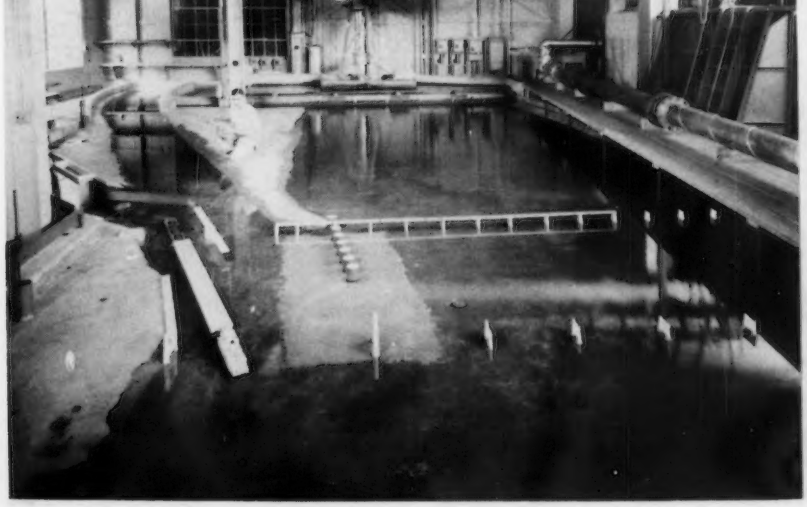


Figure 6.--MODEL DURING TEST WITH TOW

of this model are shown in Figs. 5 and 6. The river bed was carefully molded to existing topography which was obtained from hydrographic surveys. Space and cost considerations would not permit construction of the main river channel for the 2-3/4-mile reach to the dam. However, a study at the site disclosed that this would not be required since by the time the flows reach the canal entrance area they are nearly uniformly distributed across the river. It was possible, therefore, to simulate the river flow patterns by placing a head box with a uniform discharge weir 2500 feet (prototype) upstream from the canal entrance. Similarly, a uniform tailgate located 5000 feet below the entrance provided satisfactory outflow conditions. A stage recorder had been operated for many years near the downstream point of Patton Island. A study of the change in stage at this point with respect to change in discharge at the dam allowed the development of rate-of-rise and time-of-rise relationships which were used in setting up the inflow rates to the head box and the tailwater control. Fortunately, it was found that the head-box inflows could be very closely simulated by using manually set discharge increases while the tailgate required so little manipulation to produce the correct rate of rise that its operation could also be done by manual means.

Testing of the model was accomplished in two manners. Flow paths and velocities were obtained for any desired scheme by taking 3-second time exposures at any desired interval. White floats 3/4 inch in diameter and 1/2 inch deep were scattered over the water surface prior to each exposure. The length of the float path then represented the velocity, while the path represented the flow direction. In this study there were no random currents so it was not necessary to use a method which would provide the direction of the float. All of the preliminary and developmental tests were run by using the float-photographic technique. After the basic problems had been delineated and several design approaches developed, the final decisions were determined by combining the photographic data with actual observed navigational results

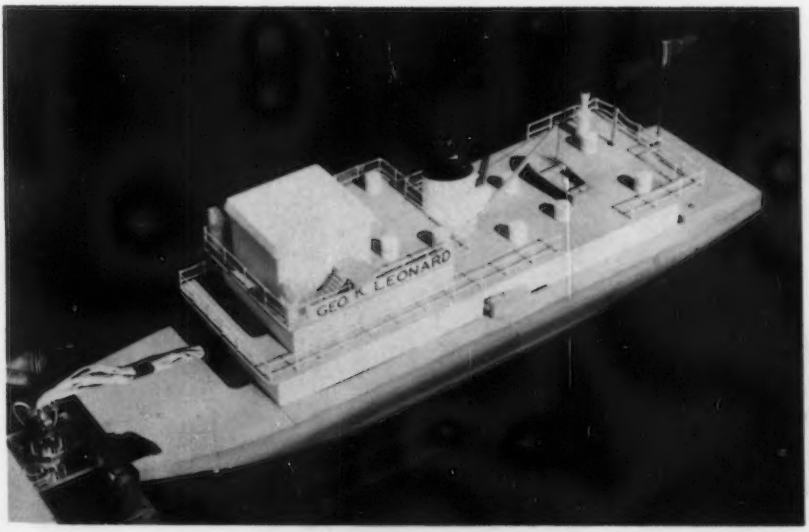


Figure 7.--1:100 SCALE MODEL TUGBOAT

obtained from the radio-controlled, self-propelled model tug and tows shown in Figs. 7, 8, and 9.

The tug, Fig. 7, was a nearly faithful reproduction of a tug which was built for Southland Towing Company by the Ingalls Shipbuilding Corporation. Available motors and control equipment were slightly too heavy to be supported in a model equivalent in size to the prototype 27-foot x 115-foot hull. Therefore, the model hull size was increased to 35 feet x 150 feet. The tug is of a twin screw construction with two steering rudders and four flanking rudders. These were all reproduced faithfully. Power to the tug was determined by adjusting the speed of the propellers until they drove the tow at a speed of 6 miles per hour (prototype) in still water. While a representative tow is difficult to define, two tows were selected for construction and testing. The small tow, Fig. 8, had a maximum width of 50 feet and was 975 feet long. The makeup of this tow was one 35-foot x 225-foot barge, two 50-foot x 300-foot barges, and the 35-foot x 150-foot tug. The large tow, Fig. 9, had a maximum width of 105 feet and a length of 1200 feet. This tow was constructed of six 35-foot x 225-foot barges in two sets, three abreast, four 50-foot x 300-foot barges in two sets, two abreast, and the 35-foot x 150-foot tug.

One of the laboratory engineers made a trip from Knoxville to Chattanooga on a similar tug to observe the manipulation and operation of an actual tug and tow. In model study observations, the model tow operator made a determined effort to operate his tow in a similar approach manner for all test schemes so that each scheme could be fairly compared with the other schemes. After some testing, the boat operator became quite proficient at determining the difficulty he experienced in propelling the tow into the canal entrance for each scheme. While tests of this type must be essentially qualitative in nature, it is felt that they add much valuable knowledge to that obtainable from the current direction and velocity observations.

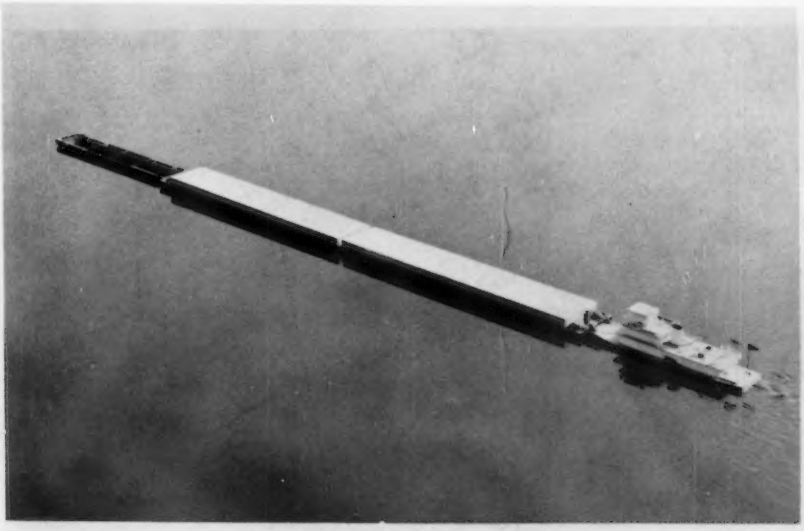


Figure 8.--1:100 SCALE MODEL OF 50' x 975' TOW

Model Results

The float studies very quickly established that the realigned Florence Canal without any special treatment at the canal entrance would be subject to the steady-flow cross-current problem and the "tidal" varying-flow cross-current one. Fig. 10 depicts the steady-flow problem. The slow eddy around the railway bridge probably would not be objectionable; however, the strong currents which would tend to act on the tow until it had passed upstream from the highway bridge would make very difficult the guiding of the front and rear of a long tow with the accuracy required to keep the tow from striking either the swing span or the bank. Fig. 11 shows the currents occurring in the original entrance 25 minutes (prototype) after a full powerhouse increase test was started. This represents about the maximum cross-current condition but only one of a nearly infinite pattern of current conditions. This figure quite nicely demonstrates the existence of a very real problem from the "tidal" effect standpoint with the restricted railway bridge swing span area subjected to most undesirable cross currents. Tests with a falling stage or "ebb tide" condition showed that the flow out of the canal passed directly down the navigation channel so that no problem whatsoever existed with this condition.

A pilot approaching the canal would, of course, be unaware of the exact current condition and could not, therefore, anticipate the effect on his tow of the existing currents. As a consequence, in the area where he would need his best control, he would be faced with an unknown and potentially serious cross-current situation. The need for some form of canal-entrance modification was thus very well proven.

The canal-entrance modification plans can be broadly catalogued into four major divisions:

1. Dikes to exclude the cross currents from the critical railway-bridge swing-span area.

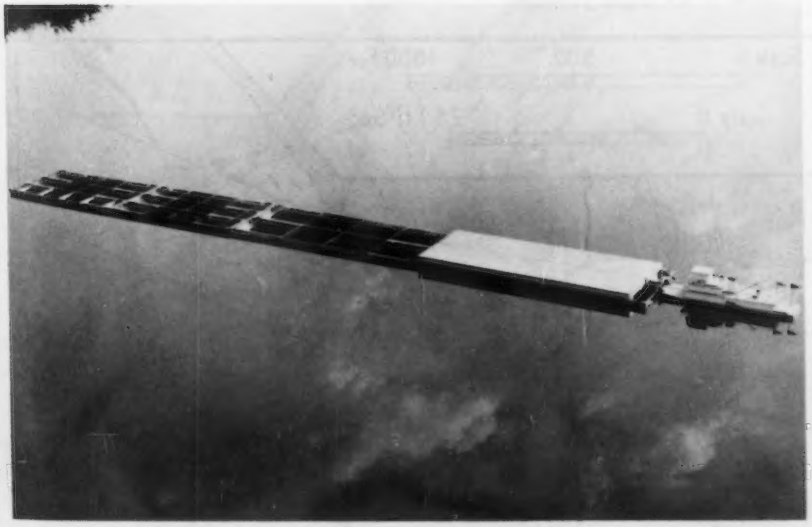


Figure 9.--1:100 SCALE MODEL OF 105' x 1200' TOW

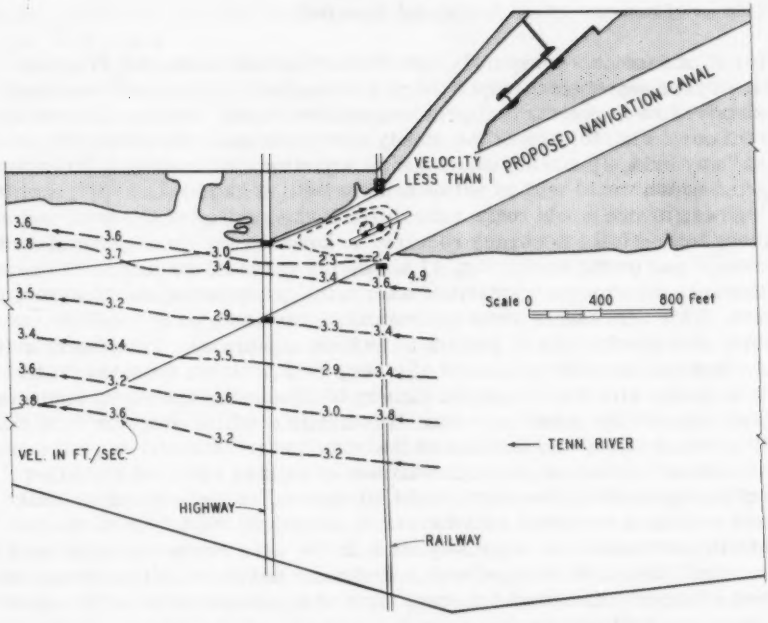


Figure 10.—STEADY-FLOW CURRENT PATHS

2. Excavation of the point of the island to move the cross currents away from the critical railway-bridge swing-span area.
3. A broadening of the island point so as to allow the cross currents to form to the left of the swing-span pier and thus produce satisfactory flow in the right passage.

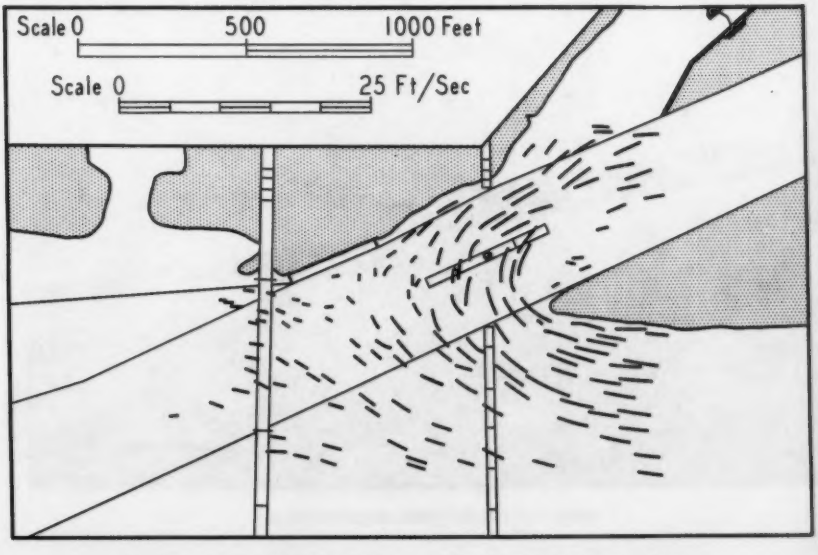


Figure 11.—UNSTEADY FLOW—ORIGINAL ISLAND

4. A broadened island point plus individually spaced sheet-piling cells located between the railway and the highway bridges.

Dikes

The dike schemes were abandoned after only preliminary testing, because any stable dike would be so wide at the base that it would create a major constriction in the main river channel with a subsequent marked increase in steady-flow cross-current velocity. Furthermore, while the "tidal" cross current could be moved away from the critical area, the dike required would be of such length as to make it economically questionable.

Island Point Excavation

Fig. 12 shows the current paths 35 minutes after the start of a full powerhouse increase test. This represents about the maximum cross-current condition. It will be noted that such a design could result in removal of the strong cross currents from the critical railway-bridge swing-span area, but only at a very considerable cost, because at least 1000 feet of the island point would have to be removed to move the cross currents adequately out of the critical area. This scheme was thus abandoned because of cost.

Island Point Extension

Broadening and extending the point of the island resulted in the operation shown in Fig. 13. This design produced a fairly acceptable design from the "tidal" current standpoint if all upbound navigation would use the right passage through the swing span. This action resulted from the broadening of the point, which caused the flows to swing in a longer arc and produced a flow in the right passage which was nearly parallel to the channel. Under steady-

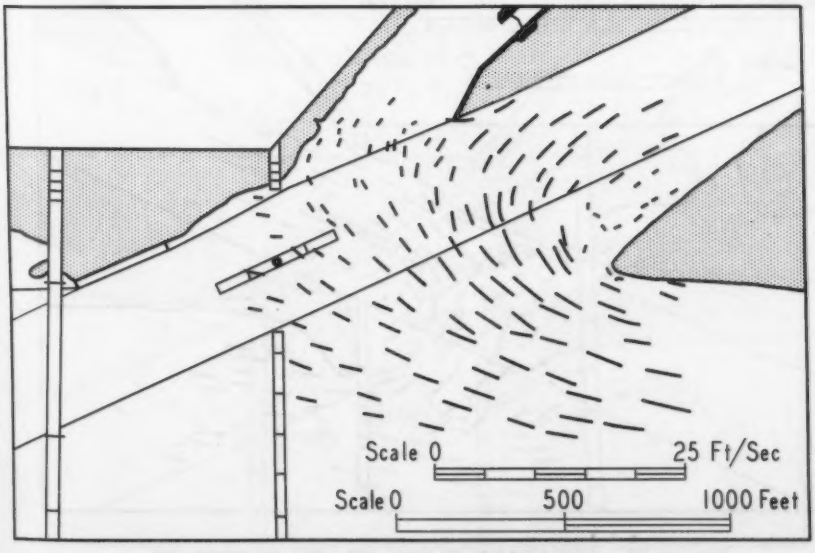


Figure 12.—UNSTEADY FLOW—ISLAND EXCAVATED

flow conditions, however, it was found that the cross-current effect was so acute that it was essentially impossible to pilot a tow through the entrance without striking either the swing span or the bank.

Island Extension Plus Cells

Addition of spaced, individual sheet-piling cells between the railway and highway bridges produced two desirable improvements to the island point extension scheme. The first was a breaking up of the current flowing into the canal under "tidal" action, with a subsequent reduction in the velocity of these currents and further straightening of the currents with respect to the channel. Figs. 14 and 15, which depict a six-cell and a nine-cell design, respectively, illustrate this action. The second desirable improvement was the breaking up of the strong steady-flow cross current in the vicinity of the highway bridge. The six-cell design was found to be slightly more desirable under steady flow than the nine-cell scheme. The six-cell design allowed some flow to pass to the right of the highway bridge pier while still effectively shielding most of the entrance area from the adverse cross currents. The nine-cell design effectively prevented any cross currents from forming in the area between the two bridges, but in doing so it increased the velocity of the cross current downstream from the highway bridge. The towboat tests showed that it was appreciably easier to enter the bridge area under the six-cell flow conditions than under the nine-cell conditions.

A considerable number of tests were run with different numbers of cells and with varying spacing of cells. Some tests used non-uniform cell spacing and some, uniform. In general, less than six cells resulted in undesirable operation, and uniform spacing resulted in better operation than non-uniform spacing.

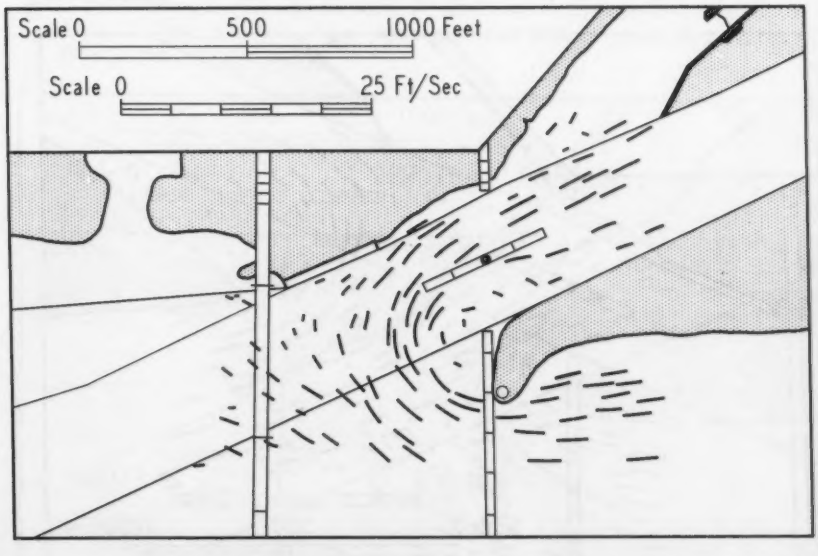


Figure 13.—UNSTEADY FLOW—ISLAND EXTENDED

Recommended Design

The final recommended and adopted design is shown in Fig. 14. The island point was extended to just upstream from the railway bridge and swung about 200 feet to the left. The design engineers did not wish to place rock fill against the existing railway bridge piers. Therefore, a single 32-foot-diameter sheet-piling cell was placed as shown immediately upstream from the railway bridge to cut off essentially any flow upstream from the bridge. Six 32-foot diameter sheet-piling cells spaced on 57-foot centers, with the upstream cell 25 feet below the railway bridge, completed the design. With this scheme the towboat tests showed that a pilot can use the same general approach to the canal for any flow condition and will find the current vagaries to be so small that he will have enough maneuvering room to pilot his tow safely through the constricted area.

Regardless of how good the current conditions could be made, piloting a tow 100 or more feet wide through the 123-foot swing span opening must always remain a precarious and doubtful maneuver. It is hoped and anticipated that, some time in the near future, the present swing span will be replaced by a lift span which will give a 350-foot clear channel. Tests with the 105-foot x 1200-foot tow shown in Fig. 9 on the recommended scheme proved that with the center swing span pier removed, such barge tows could be brought into the canal with no trouble.

ACKNOWLEDGMENTS

These tests were started during the period when Mr. C. E. Blee, M. ASCE, was Chief Engineer for TVA. They were completed under Mr. George K. Leonard, M. ASCE, present Chief Engineer. The model studies were performed under the general supervision of Mr. Albert S. Fry, M. ASCE, and the

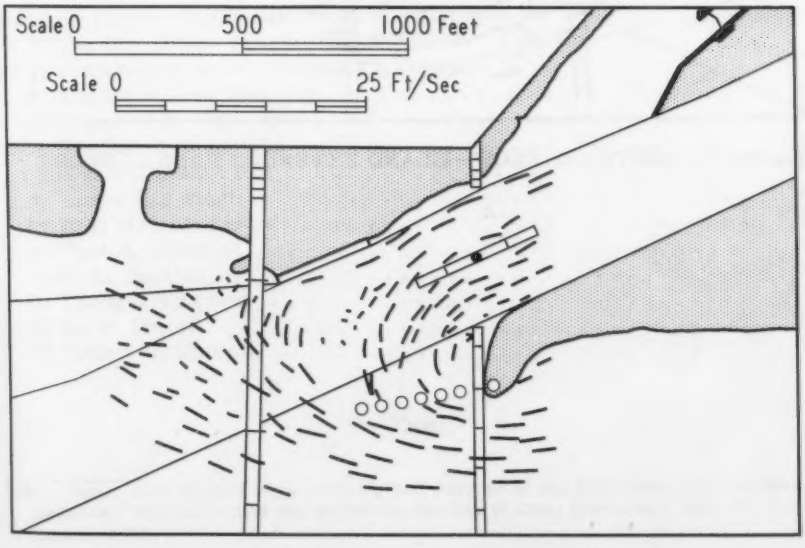


Figure 14.—UNSTEADY FLOW—ISLAND EXTENDED PLUS 6 CELLS

immediate supervision of the author. Design features were coordinated within TVA with Messrs. R. A. Monroe, M. ASCE, G. P. Palo, M. ASCE, and W. F. Emmons, M. ASCE, and navigation features with Messrs. J. Porter Taylor and R. B. Nichols, Assoc. M. ASCE. Mr. C. T. Capp, general sales manager, Barge and Towboat Division, Ingalls Shipbuilding Corporation, supplied the designs for the towboat. The A. L. Mechling Barge Lines, Incorporated, of Joliet, Illinois, allowed the engineer from TVA's Hydraulic Laboratory to ride their towboat, Elisha Woods, so as to become acquainted with operation of a tow. Captains J. D. Jones and O. A. Franks of the Elisha Woods were extremely helpful in explaining the operations.

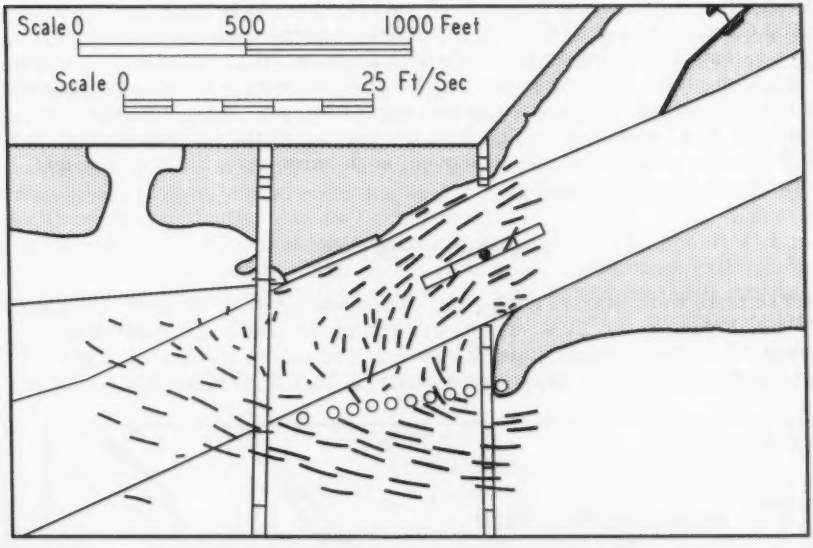


Figure 15.—UNSTEADY FLOW—ISLAND EXTENDED PLUS 9 CELLS

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

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SHIPBOARD HYDRAULIC BREAKWATER[a]

Closure by Russell A. Dilley

RUSSELL A. DILLEY.[1]—Mr. Herbick's remarks on the measurement of wave heights are well taken. In the tests it was found that due to boundary conditions imposed by the limited size of the model test area severe reflections resulted after only a few waves had been attenuated by the jet action. For this reason, it was felt that only these few undisturbed waves were typical.

a. Proc. Paper 1569, March, 1958, by Russell A. Dilley.
1. Engr., Univ. of California, Institute of Eng. Research, Berkeley, Calif.

FORCES INDUCED ON A LARGE VESSEL BY SURGE[a]

Corrections to Discussion by Basil W. Wilson

CORRECTIONS TO DISCUSSION BY BASIL W. WILSON,[1] M. ASCE.—In Proc. Paper 1884 (December 1958 Journal of the Waterways and Harbors Division) the following corrections should be made to the discussion by B. W. Wilson (pp. 9-15):

(a) On p. 11 the line above Eq. (14) should read ". . . the values of u and $\dot{u}$ at $x' = L/4$ . . ."
(b) Eq. (14)(ii) should read "$\dot{u} = \ldots$"
(c) On p. 14 near the bottom of the page, Eq. (31) should be amended to (32).
(d) On p. 14, the fifth line from the bottom of the page should read "Eq. (32) can also be written"
(e) On p. 14, near page bottom, Eq. (32) should be amended to (33)
(f) On p. 14, the second line from bottom should read ". . . right hand side of Eq. (33) . . ."
(g) On p. 15, top line, Eq. (32) should read (33)
(h) On p. 15, third line, Eq. (33) should read (34).

Also, Mr. Wilson's membership status should be changed from A. M. ASCE to M. ASCE.

a. Proc. Paper 1571, March, 1958, by J. T. O'Brien and D. I. Kuchenreuther.
1. Associate Prof. of Eng. Oceanography, A & M College of Texas, College Station, Tex.

STABILITY OF COASTAL INLETS[a]

Closure by P. Bruun and F. Gerritsen

P. BRUUN,[1] M. ASCE and F. GERRITSEN.[2]—The problem of stability of tidal inlets includes, as stated by the authors, many factors which may be permuted in different ways. In regard to the two problems named by Mr. Hickson, the Columbia River and the channel which connects Lake Maracaibo with the Gulf of Venezuela, it is obvious that such conditions cannot easily be included in a "formula". What we, in studying these problems, are trying to do, therefore, is to analyze and try to understand the simple problems first. Before this has given results, no chance exists for an understanding of the more "difficult" problems.

In his commentary Mr. Hickson illustrates the variation in ratio factor between the tidal prism and the cross-section for the Columbia River under various river discharge conditions (head flow). It is interesting to note that similar variation in the value of the ratio factor exists for tidal inlets without head flow, but with a variation in tidal conditions in the sea. Examples of such inlets can be found on the Florida Gulf coast where the tide is a mixture of diurnal and semi-diurnal. In all such cases the analysis should be based on the time history of the shear stress along the wetted perimeter of the gorge. The following definition of τ_s may be practical:

$$\tau_s = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} |\tau| \, dt$$

The time interval $t_2 - t_1$ should be long enough for profile adjustment. In determining τ_s from the above equation it should be remembered that the shear stress at the bottom is proportional to the square of the "bottom velocity", which has to be known.

For practical reasons the authors, in line with O'Brien's approach of 1931, assumed the mean spring tide conditions to give a close approximation to the determining values.

Even in the "complex cases" it is considered only right to bring the problem of stability back to forces exerted by the flow on the bottom. Otherwise a rational understanding cannot possibly be achieved.

In case of rivers the difference between bottom velocities during ebb and during flood may necessitate special treatment of the case, but bottom shear stresses, being the direct cause for transport of bottom material, will still

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a. Proc. Paper 1644, May, 1958, by P. Bruun and F. Gerritsen.

1. Director, Coastal Eng. Lab., Univ. of Florida, Gainesville, Fla.
2. Research Eng., Coastal Eng. Lab., Univ. of Florida, Gainesville, Fla.

determine the stability of the bottom whether or not they are caused by salt, brackish, or fresh water.

In regard to the channel at Venezuela, the water-body between the bar at the Gulf of Venezuela and the Strait of Maracaibo which is the entrance to the main body of the lake is called Tablazo Bay. Tidal and current observations have demonstrated that practically all of the water which causes the tide to rise and fall in Tablazo Bay may come from the Strait of Maracaibo, rather than from the Gulf of Venezuela which, in turn, means that a nodal point for the tidal current exists someplace in Tablazo Bay. Perhaps said nodal point can be considered as a node for a seiche in Lake Maracaibo (information by G. A. McCammon). The existence of density currents is also clearly demonstrated in the Strait of Maracaibo where salt water inflow is predominant at greater depths and fresh water outflow in the upper layers. In the dry season flow is mainly lakeward; in the rainy season mainly seaward. Averagely the flow is seaward for nine months and lakeward for three months. In a case like that the stability is still determined by shear stresses but they have to be measured under the actual current conditions which depend upon a variety of factors which have no simple relation to the actual tidal prism and must count on the existence of density currents also.

Since the printing of Paper No. 1785 in the ASCE Proceedings, further progress in our studies has been made, the most important result being that the determination that the tidal prism Ω should be replaced by the maximum flow Q_m. Again in determining the value of Q_m, the time history of the flow should be considered during a period long enough to allow adjustment of the profile to the actual flow conditions. Proper values for τ_s should be determined with different sediment load, waves, bottom material characteristics, etc.

A paper on the results of these investigations is being published by the Florida Engineering and Industrial Experiment Station and can be obtained by those interested by writing to the Coastal Engineering Laboratory of the University of Florida (Building L) Gainesville, Florida.

We appreciate Mr. Hickson's remarks which give examples of complex problems which still remain to be solved, as well as our private correspondence with Mr. Hickson, who as earlier member of the Tidal Hydraulics Committee of the Corps of Engineers has preserved and demonstrated a great interest in the "stability of coastal inlets".

THE SUEZ CANAL—ITS CHRONICLE AND BIBLIOGRAPHY[a]

Discussions by Woodland Gray Shockley, Raymond A. Wheeler, Frederick K. Morris, Jos. Millecam, James Kip Finch, John William Leslie, Paul A. Blanquet, R. L. Bostian, Jr., Tsung-Lien Chou, W. E. Potter and Arthur B. Cleaves

WOODLAND GRAY SHOCKLEY,[1] M. ASCE.—The paper is extremely interesting and gave the writer much pleasure in reading about some of the lesser known incidents that accompanied the building and subsequent operation of the canal. It was a little difficult to follow the ancient courses of the canal without at least a sketch map. Perhaps the author could include one in his closing discussion. The writer thinks it would add to the paper. Also, it would be very valuable if an account could be added of the restoration of the canal following the fighting a couple of years ago. This in itself ranks high in engineering works and would be a fitting climax to the author's very valuable paper.

RAYMOND A. WHEELER.[2]—The writer considers it a valuable document on a subject that has his great interest and attention. While the writer does not feel well enough to prepare a more thorough discussion, he would like to express that he has read it with intense interest and considers it a superb presentation. The author is to be congratulated for a superior job. The writer hopes to see the paper the author is preparing for the period subsequent to nationalization as that is the period, of course, with which the writer has had his principal duties in clearing the blockade under the auspices of the United Nations.

FREDERICK K. MORRIS.[3]—Fig. 1 will help to clarify questions concerning the geology of the Canal route. The data are chiefly from Krenkel's "Afrika" —unfortunately, incomplete and not up-to-date—and from Andree's "Hand-atlas". It is advisable to get later information than is here summarized.

The northern half of the canal extends from Port Said to Ismailia at the fresh water lake Timsah, through an almost flat country of silt and sand.

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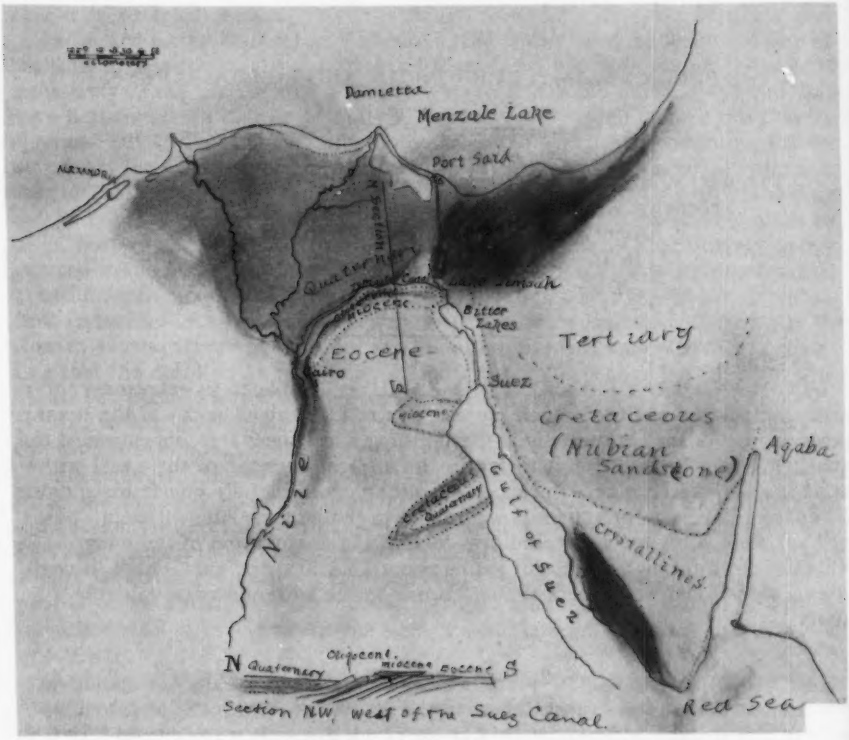
a. Proc. Paper 1770, September, 1958, by Shu-T'ien Li.

1. Asst. Chf., Soils Div., Waterways Experiment Station, Corps of Engrs., U. S. Army, Vicksburg, Miss.
2. Lieut. Gen. (Retired), U. S. Army; Cons. Engr., International Bank for Reconstruction and Development, Washington, D. C., formerly Chief of Engineers, U. S. Army.
3. Emeritus Prof. of Eng. Geology, M.I.T.; Cambridge, Mass.



Westward this plain merges into the Nile Delta, and eastward into the dry coastal plain of Palestine.

The southern half of the canal extends from Lake Timsah to Suez. The Bitter Lakes are remnants of the Gulf of Suez—which even within history extended to include them. From the Bitter Lakes to Suez low chains of hills enclose the Canal and rise into gently rolling plateaus of sedimentary rocks, which the writer summarizes:

QUATERNARY—loose delta and desert deposits.
-----Disconformity.
PLIOCENE—heavy gravels, sands and silts.
MIOCENE—chiefly fresh water sands and clays.
-----Conformity.
EOCENE—chiefly marine limestones covering large areas. The stone for the Pyramids of Gizeh was quarried from this rock, and the Sphynx is a natural hill carved (and even restored) into its shape.
CRETACEOUS—chiefly the Nubian sandstone and shale.

Some areas of the Cretaceous and Eocene are gently folded and oil prospects are connected with them.

In the Sinai Peninsula ancient crystalline rocks include schists and granites.

JOS. MILLECAM.[1]—Being a former member of the "Commission Internationale Consultative des Travaux" of the Universal Company of the Suez Canal, the writer is naturally very much interested in the paper and is in a position to appreciate it. He has read it with great interest and wishes to congratulate the author for the comprehensive way in which he has expounded the formidable task carried out by Ferdinand de Lesseps and his collaborators and the tremendous work of progress and prosperity achieved by the former Universal Company of the Suez Canal.

Hereunder are a few observations the writer wishes to raise in connection with the paper:

(1) More should have been said about the particularly arduous technical problem the Company had to deal with in order to protect the Canal banks from erosion caused by the passage of large vessels. Two remarkable papers were presented on this question; viz:

(a) At the 17th International Navigation Congress (Lisbon 1949) by Mr. Max BAHON, Directeur Général honoraire, Vice-Président du Conseil d'Administration de la Compagnie, Paris (Communication 2 of Section I—Inland Navigation).

(b) At the 18ths International Navigation Congress (Rome 1953) by Mr. LEROY, Ingénieur des Ponts et Chaussées, sous-Directeur de l'Office de la Navigation, Paris (Communication 1 of Section I).

(2) The bibliography should also mention the two papers referred to under par (1) above.

The bibliography should mention the remarkable book published in Paris in 1951 (Printers: Plon Library; Editor: 8 rue Garanciére, Paris VI) and entitled: "Ferdinand de Lesseps—Le Diplomate—Le créateur de Suez", by George Edgar-Bonnet, Honorary Director General of the Suez Canal Company.

This book gives a detailed and captivating description of Ferdinand de Lessep's life, namely about his tenacious strife to overcome opposition against the principle of excavating the Canal of the "Sublime Porte" (sultan of Turkey at Constantinople who was suzerain to the Viceroy of Egypt) and mainly about the almost inexplicable opposition of the United Kingdom Government (Lord Palmerston—Prime Minister).

It called for resolute conviction and steeled energy not to be overcome in these struggles, in spite of the little support granted to Ferdinand de Lesseps by the then French Government.

The life of men of that calibre deserves to be known and appreciated by all civilised peoples.

(3) Hereafter are a few minor observations:

(a) Page 1770-2—lines 12 and 13:
"Thus it constitutes the life line of Western European maritime countries, particularly Britain and France".

One should also add: "the Netherlands" which country kept up an intense maritime traffic with their East India colonies (at present Indonesia).

1. Honorary Secy. Gen. and Technical Adviser of the Permanent International Assn. of Navigation Congresses, Brussels, Belgium; formerly member of the "Commission Internationale Consultative des Travaux", Universal Co. of the Suez Canal.

(b) Page 1770-2—line 18:
"... the French Engineer, Ferdinand de Lesseps ..." de Lesseps was not an engineer, but a diplomat.
(c) Page 1770-8—line 14:
One should read 1858 instead of 1958.

JAMES KIP FINCH,[1] M. ASCE.—In this interesting historical-chronological summary the author quotes from the early—about 60 B.C.—account by the Greek writer, Diodorus of Sicily, of ancient canal constructions between the Nile Delta and the Red Sea. He has, apparently, followed a translation similar to that of C. H. Oldfather in the Loeb Library Version of Diodorus which states that, "in the most suitable spot," the Second Ptolemy "constructed an ingenious kind of a lock." Since locks were not invented until some 1500 or more years after Ptolemy had passed on the use of the word lock in this translation is evidently incorrect. Actually Diodorus referred to the device in question as a diaphragma, obviously the origin of our word diaphragm, which is properly translated as a partition or barrier and cannot have meant "a lock in a canal." Clearly it would require two diaphragma to form a lock and, while a water gate may not have been impossible at this early date, a barrier, probably of stop logs, seems a much more plausible supposition.

One of the earliest references to improvements in navigation of the canal or canalization type has to do with the first cataract of the Nile at Aswan—700 miles up the river. Here one, Uni, was commissioned to open a passage for boats about 2500 B.C. It has been assumed by some writers that in this case also a lock was necessary and accordingly one was constructed. Actually all that was probably done was to remove some boulders or projecting ledge so that a boat could be pulled through against the swift current of the river.

Over twenty centuries later, in the papyrus letters and orders of a Greek architekton, Kleon, which have survived from the Fayoum in lower Egypt, and date from the same Ptolomaic period as the Nile-Red Sea canal, reference is made to closing sluice openings by stop-logs. Still later, in the reign of the Roman Emperor, Trajan, about 115 A.D., there is interesting correspondence with his friend and governor of Bithynia (Turkey) Pliny the Younger, in connection with a canal near the city of Nicomedia. It is clear that not only were locks unknown at this time but that the dangers of attempting to control flow by the means—stop-log or other—then available, were also appreciated. Trajan warns Pliny not to connect his proposed canal to an inland lake lest the lake be drained. Pliny replied he would leave a strip of land between the end of the canal and the lake.

Far to the East the Grand Canal of China, extending from Hangchow to Tientsin and thence by river to Peking—800 miles—is usually accepted as having been a growth rather than a single project and to have been begun possibly as early as 600 B.C. but not completed until some nineteen centuries later. But the ingenious Chinese, even with this long experience with and use of canals, never devised a canal lock. Either one changed boats at changes in level or stop-logs were removed and the boat pulled through against the current with ropes.

1. Dean Emeritus & Renwick Prof. of Civ. Eng., School of Eng. Columbia Univ., New York, N. Y.

Canal locks were clearly not invented until the late Middle Ages or Early Renaissance. It has been suggested that they appeared first in Holland while other authorities argue that they were first developed in Northern Italy. It seems quite probable that tide-gates did originate in Holland and it is certainly not impossible that locks were developed independently in both countries. It was in Italy, however, that the European canal era was born as is interestingly revealed in Italian records.

While the lock is a strictly navigational device the Italian canals were not originally designed to meet transportation needs. As early as 1167 Milan and other northern Italian cities had banded together in the Lombard League for mutual defense. Milan's new defenses involved a moat and water for this was diverted from nearby streams. The idea of also using such water for irrigation soon developed. Additional supplies became necessary and the city turned for this purpose to the Ticina River, 16 miles away, and called the canal the Ticinello. Through such growth a net-work of canals developed in the valley of the Po and its tributaries. In time, these canals were used to transport materials and produce and the Ticinello became known as the Naviglio Grande. These canals were all of the river canalization type. The first canal to cross a summit was the French Canal de Briare of 1642.

As early as the 12th Century conche, or basins, formed by low dams had been used in Italy to hold back the flow on excessive slopes and divert water to side ditches for irrigation. Apparently hand-operated sluice gates early replaced the stop-logs which were probably first used to control such flow. General Parsons in his study of Engineers and Engineering in the Renaissance, reproduces an interesting drawing of early but uncertain date which shows a river civilization with a series of quite elaborate vertical life gates operated by capstans. The idea of the lock must have become immediately evident when two such gates were placed close together. Thus the two lower gates in this drawing appear to have been operated to form a lock basin or chamber. The lower gate holds back the water to the level of the upper reach and the boat passing through the open upper gate does so without encountering any adverse currents.

Apparently, however, the first known description of a canal lock is that contained in the first printed book on architecture, the De re aedificatoria of Don Battista Alberti, published in 1485 but probably written about 1450. By this time locks were well developed and other forms of gates had given way to those turning horizontally on vertical hinges. It is also possible that Alberti had in mind another essential device, the wicket gate used in the main lock gate to empty or fill the lock chamber and relieve the pressure on the larger gate. In any event the note books of Leonardo da Vinci (1452-1519) show not only balanced wicket gates but also the mitering lock gates which have become the standard form for lock use.

Turning back to Diodorus one also notes that he states that Darius the Persian some 500 years B.C. had been advised—and erroneously as was Napoleon some twenty five centuries later—that the Red Sea was higher than the Nile and Mediterranean. Actually, as Dr. Li notes, there is no appreciable difference in level and no locks were required on the Suez Canal. One wonders, therefore, why Ptolemey the Second found it necessary to provide even a diaphragma for a canal from the Nile to the Red Sea. Possibly it was used to keep salt water from entering the canal for it is probable this canal served not only for navigation but also for domestic water supply and irrigation.

A lock with many gates. From the *Codice Laurenziano* (Laurentian Library, Florence).

JOHN WILLIAM LESLIE.[1]—The subject and the author's presentation of it is very interesting to the writer, principally because of a desire to compare the Suez Canal, and its problems, with our sea-level Cape Cod Canal in New England. Of course, there are major differences between the two, but there are also many common problems.

The writer would like to call to the author's attention the 1945 Corps of Engineers study of a proposed sea-level Panama Canal. In this report printed in 12 volumes, there is a rather interesting study and comparison of all major canals, including the Suez and the Cape Cod Canal, particularly in those

1. Chf., Eng. Div., U. S. Army Engr. Div., New England, Corps of Engrs.

features which would be indicative of the problems and characteristics to be expected of a sea-level Panama Canal.

As stated in the Synopsis, a later paper will deal with economic aspects of the Canal, and its immediate past and probable future. Actually the economic factors are more complex and unfortunately more important than the engineering problems in determining whether these Canals, or any of our waterway projects, will be further improved or maintained. The writer will be very interested in reading the author's views on the economic aspects of the Canal and in particular his views on whether the Canal can keep pace with the rapid increase in size of modern tankers, or whether that commerce will go outside, and the Canal lose the great bulk of its present commerce.

PAUL A. BLANQUET.[1]—Mr. Shu-t'ien Li's "Chronicle" constitutes, from the technical point of view, a condensed but extremely rich and well documented history of the Suez Maritime Canal.

After having recalled the former existence of navigable waterways which, in Antiquity, linked the Gulf of Suez to the River Nile, the author has successfully brought out the tremendous significance of Ferdinand de Lesseps' undertaking—so outstandingly audacious for the nineteenth century—which opened to the ocean-going fleets a direct and sea-level passage from the Mediterranean to the Red Sea. He has most competently demonstrated how the Concessionary Company, over a period of nearly 87 years, perfected this great work of "continuous creation" in order that it should always keep pace with the evolution of transiting vessels whose number, tonnage, and speed were perpetually increasing. Finally, he has shed the appropriate light on the importance of the program of works which, in July 1956, was being undertaken or planned by the Universal Company to fully adapt the Suez Canal to the future requirements of world shipping.

Economists as well as engineers will find the documentation thus assembled in Mr. Shu-t'ien Li's "Chronicle" of the greatest interest and will undoubtedly be grateful to him for appending thereto a chronological bibliography of the previous main technical publications which have been printed on the subject of the Suez Canal.

It goes without saying that the very scope of the Paper permitted the author to sketch out only in their broad lines a number of the problems with which the engineers of the Universal Company were confronted during approximately a century and to which they had to find a solution. Among the more recent of these two, which are of considerable concern to present day builders of large maritime works, appear worthy of further emphasis.

The first of these problems bears on the relationship between the cross-section of a canal and certain characteristics of the vessels intending to use it—such as draft, width, and speed. Mr. Shu-t'ien Li, on p. 30 of his Paper, has been kind enough to refer to the experiments carried out by the writer, in this connection, both in the "Bassin des Carenes du Ministere de la Marine" in Paris and in the Neyrpic Hydraulic Laboratories in Grenoble and which were the subject matter of an account he presented to the American Society of Civil Engineers in February 1955, (Vol. 81—Proceedings Separate No. 613).

Since then a detailed report on these important experiments has been published in the French review "Annales des Ponts-et-Chaussees" (Nov. 1956-

1. Ingénieur-en-Chef Honoraire de la Compagnie Universelle du Canal Maritime de Suez, Paris, France.

Feb. 1957) by M. M. Suquet, Barbier, and Gamot, who are all engineers of the Grenoble Laboratories. Although it would not be possible to list here all the results of this detailed study, it is believed that some interest will attach to setting forth the principle conclusions that can be drawn from it.

1. The influence of the variation of a given parameter, such as the speed of a transiting vessel, its draft or the configuration of its immersed section, becomes all the greater as one draws nearer to the limiting factors of possible transit. Thus, in a specified section, a transiting vessel characterized by the dimensions of its mid-ship frame would not be able, practically speaking, to exceed a limiting speed. The experiments, therefore, confirm both qualitively and quanitively the remarkable theoretical studies put before the International Congress of Navigation of Lisbon in 1949, by the Dutch engineer, Mr. Schijf.
2. The curves representing hydraulic phenomena and energy consumed by reference to the relationship between the cross-section of the Canal and that of the mid-ship frame of a transiting vessel present a notably hyperbolic aspect.
3. The intensity of the hydraulic phenomena and the power absorbed by the screw largely depend upon the shape of the cross-section. In practice, the banks vary according to the nature of the terrain and this, in the Isthmus of Suez is very inconsistent. The implication is that in order to attain fairly consistent results from a hydraulic point of view it is necessary to plan cross-sections whose surfaces, depending on the area, may vary from 7 to 8%.

Agreeing, as they do, with the results of direct observation programs and with measurements taken in the Canal itself, these laboratory studies have made it possible

- on the one hand to establish as correctly as possible the regulation transit speeds so as to avoid deterioration of the banks and facings;
- on the other hand to select the most appropriate prototypes of cross-sections suitable to the various sectors.

The second problem to which it is felt that attention should be drawn relates to the most recent forecasts of traffic—evolution made by the Concessionary Company and to the conclusions they inspired.

Advance computation of the numbers of ships that will require accommodation in the future and of the dimensions of the largest of these has always been one of the objectives of the Concessionary Company whose successive improvement programs have, needless to say, been predicated on such forecasts. Thus, in particular, the Eighth Program of improvements—which was still under way in 1956—had been drawn up in 1954, on forecasts of traffic-evolution that were valid in 1953.

It fairly soon became apparent, however, that these relatively generous forecasts might run the risk of being out-paced by the extraordinary increase in the requirements of the petroleum carrying traffic. Consequently, the Concessionary Company, as early as the end of 1955, undertook a revision of its forecasts with a view to bringing about a Ninth Program upon which it intended to embark even before completion of the Eighth.

It is thus that early in 1956, the Universal Company—as mentioned by Mr. Shu-t'ien Li (p. 21 of his "Chronicle")—signed a contract with the American firm "EBASCO Services Inc." on the terms of which these engineers were

to undertake a comprehensive study, carried out with the most modern methods of investigation, of the traffic foreseeable until 1972, i.e., until three years after the normal expiry date of the Company's concession.

Ebasco Services Inc. based its study on the following six assumptions:

1. There would be no widespread wars or extensive political revolutions during the period.
2. Political and economic conditions in the world would show gradual improvement.
3. The policies of the under-developed nations would be influenced by a desire to develop their resources and their industry for the benefit of their domestic economy.
4. Credits to various countries from international agencies as well as from the United States would gradually increase in size and the difficulties facing international flow of capital would decrease.
5. Normal operation of the Suez Canal would not be substantially affected by military operations or by other hazards unconnected with the operation.
6. The flow of oil from the Middle East would not be subjected to any major interruption.

Obviously, such optimistic assumptions for the world's economy could only lead to forecasts of maximum traffic and the works to be carried out on such premises would, therefore, prove amply sufficient.

The main conclusions of the EBASCO Report were as follows:

1. Petroleum traffic which was 69 million tons in 1955, would reach 335 million tons by 1972 and the tonnage of all other goods would increase during the same period from 39 million to 54 million tons.
2. The average size of tankers using the Suez Canal would increase from 18,900 to 32,100 dead-weight tons and their average speed from 14 to 16.5 knots.
3. The daily average number of transiting vessels would rise from 40 to 86—the component represented by tankers increasing from 22 to 61 units.
4. The Canal should be made ready to accommodate tankers drawing 39 feet and having a dead-weight of about 50,000 tons.

Such are the factors upon which the Suez Canal Company based its project for a Ninth Program of Works and obtained its approval from its International Consultative Works Commission. In the main, this project consisted of:

1. The creation of an uninterrupted two-way traffic section between Port Said and Km. 66 (two separate channels were already in existence between Kms. 51 and 61).
2. The creation of an uninterrupted two-way traffic section between Kms. 95 and 134.
3. Overall enlargement of the Canal proper. In the one-way sections this was to be brought to a guaranteed depth of 49 ft., the width, at that depth, being 305 ft. in areas where the bank gradient reached 3/1. In the two-way traffic sections, either two parallel channels would be created each having the above-quoted dimensions, or a single but very wide channel permitting vessels to cross whilst underway (in principle, this channel was to be 820 feet wide at a depth of 49 feet).

4. The establishment to the east of Port Said of a new direct outlet into the Mediterranean for the northbound vessels and the creation in Suez Roads, of a second channel to avoid convoys having to pass one another in this area.
5. The construction of a tunnel underneath the canal to replace the bridge at Km. 68.

Works on the Canal itself were to involve the removal of 90 million cubic yards of dry extractions and of 400 million cubic yards of dredging. They also involved the construction of 75 miles of bank facings and the total estimated cost was 100 million Egyptian pounds ($287 million).

The foregoing data will, it is hoped, usefully supplement the information supplied by Mr. Shu-t'ien Li's Paper on this comprehensive and ultimate project of the Universal Company of the Suez Maritime Canal.

R. L. BOSTIAN, JR.[1]—It is a very appropriate time for such a paper elucidating understanding and appreciation of the problems and struggles pertaining to the Suez Canal. It is indeed a very important seaway. A better understanding of the influence and significance that this seaway has upon the economics and political actions of all concerned nations should bring light upon the psychological attitudes of the nation in which it lies. The Suez Canal should demand a high level in the interests of governmental diplomats as well as engineers.

The bibliography is international in scope. It would be of interest to know depositories in the U. S. where all or a large number of these references can be found. What percentage could be expected to be found in the library of the average engineering school? The writer may mention that the Encyclopedia Britannica has a notable entry on the Suez Canal.

To comment on some of the technical aspects of the canal, the silting problem is the most outstanding. It seems that study and research on the silting problems could offer a very challenging undertaking. Longgood's "Suez Story" mentions the fact that the Canal Company had taken steps along this line. The Company had a scale model replica of the canal built in Grenoble, France. The model was 210 feet long and 29-1/2 feet wide. Ship models measuring twenty-six feet and weighing three tons were used in experiments to plan counter measures against bank erosion. To minimize the effects of wash erosion, the engineers have walled the banks with masonry and interlocking steel piles. Other bank structures include carpets made of asphalt poured over a wiremesh frame. These carpets are laid along the banks and nailed down with ten-foot steel spikes. The feasibility of stabilizing the slopes by grout offers possibility for study. A study of the spongy material called "slob" encountered in the excavation of the Menzaleh basin and the encounters met during excavation through the sands of the El Guiser plateau, could promote an understanding of the instability of the soil structure. Subjects for consideration would be: The determination of the wave generating properties of ships using the canal. The ship's hull moving through the water generates waves by a plowing or furrowing action which cuts the water apart, creating a rising of the water at the bow, continuing as a series of undulations, partly along the hull and extending outwardly from it at an angle so that the hull appears to be traveling at the apex of an inverted V. These waves have

1. Sr. Asst. Engr., Atlantic Coast Line R.R., Wilmington, N. C.

sharp, high crests, deep troughs, and are relatively short between crests. A depressive action pushes the water down and under the hull, creating swells with low rounded crests, wide shallow troughs, relatively far apart between crests. The lines of crests and troughs are nearly at right angles to the course of the vessel. The magnitude of the waves produced is a function of the ship's velocity, displacement and the depth of the water. The determination of the forces exerted by waves involve such wave factors as the height before breaking, its velocity at the time of breaking, the relation of the wave height to the depth of water in which it breaks, the relation of the trough and the crest to the undisturbed surface, the ratio of the height to the wave length, the effect of decreasing depth upon the wave velocity, the character of wave impact and the angle of wave approach to the shore. The study of soil mechanics and the stability of the types of soil found throughout the traverse of the canal would require an investigation of materials and methods for stabilizing the soils and protecting the banks, and an examination of the causes of silting as produced by the physical condition of the canal. Factors involved could include stability properties of the soil, location of sand banks and bars, tidal effects and current effects.

The economics of traffic flow as affected by the hydraulics of open channels offers an unlimited field for research. The canal section should satisfy such conditions as to guarantee safe navigation, involve low annual or capitalized costs, offer low resistance to ship movements, and allow for future enlargement. Other factors include the determination of the most economical and safest cross section, maximum velocities allowable, most economical spacing of passing areas, and the traffic flow pattern both present and potential.

Political aspects of the Suez Canal are infinite in scope, and volumes could be written from many angles. At the present time, the canal has proved to be a political dynamite. It has been used in acquiring political advantages in the recent premature nationalization of the canal. It has long been recognized that whoever controlled the canal would have maritime supremacy. It is of advantage to the world to keep the canal in international control, and all political influence should be directed toward this end.

The author is to be highly commended for this most timely contribution towards authentic documentation of the Suez Canal. His paper reflects exhaustive research and study. The coverage is extensive for such a brief chronicle.

In preparing this discussion, acknowledgment is due to all the authors of the following references:

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TSUNG-LIEN CHOU,[1] M. ASCE.—This paper has revealed the following interesting points:

1. Natural Selection of Route and the Human Efforts. For more than 5,000 years, the present route of the Canal connecting the two massive water bodies has constantly attracted the attention of the world, simply because of the natural location and the pertinent facilities offered, with the Red Sea served as an indicator from Nature. However, it took the prominent genious of Napoleon to make a loud proposal in the time of his desperate struggle for military glory and another more persistent effort of Ferdinand de Lesseps to realize the dream of years. If there had not been such two figures, the Canal would have been delayed for some time, but not until present day. The more sane problem is: Is there any alternate route possible to avoid the various difficulties experienced in the present one? This may have been considered in different quarters at different times and so far, no answer has been known. Yet, it will remain open in future.

2. Evolutionary Growth of the Structures with the means of Maritime Transportation. With the advance of science and technology, the ships have been incessantly modified in sizes, equipments, and proportions by the improved methods of propelling instruments, fuels etc. Thus, no one can be justified to blame the moderate scales started by the original design and, yet, no one can envisage any limits of growth, if the Canal can survive some future innovations. But one point stands out in this record as a very instructive lesson for engineers: structures of international standing should be designed with generosity rather than with mediocrity, unless there is sound basis to be otherwise.

3. Heavy Maintenance and Improvement. In the past nearly 90 years of existence, the volume of earth removal for maintenance and improvement comes out to be more than 6 times of the original excavation. This fact deserves close attention. The chief trouble comes from the desert region. The earth there is sandy and unstable by itself in the disturbance of viscous fluids. Further, proximity to the muddy Nile and the Sahara Desert brings in material from a great distance. It is evident that any scheme to bring down the recurring perennial expense requires the extension of the Canal jurisprudence beyond its present periphery both physically and categorically.

4. Interweaving of Multi-purpose Developments. To prevent inroad of eolian erosion and deposition of fine silt, palliative belts of shrubs, or other measures of soil conservation, as often used in Lowland Countries of west European coasts, are necessary. This may be started in the neighborhood and extended gradually in the windward direction. Of course, irrigation and land reclamation, as done in recent years on the west side, will also give relief to the Canal maintenance. By extensive cultivation, the local climate can be moderated. But here another point of importance comes in. The agriculture of this swampy and sandy area can hardly compete with the alluvial Nile valley, unless a more lucrative market is created by encouraging some industry. Next, let us consider the silt from the very turbid water of the Nile

1. Hydr. Eng. Bogert and Childs, Cons. Engrs., New York, N. Y.

effluence which is induced by the gentle movement into the Red Sea. Here, the possible solution will be some desilting mechanism, such as a settling pool or bay with some coagulants. Before any proposal seriously considered, scale model tests will offer reliable solutions in various ways. In a word, if the Canal structures are kept up to its mission in the modern sense, the required activity should include some social functions.

At the present moment, the chief cargo through the Canal is the petroleum products which are in demand by Europe from the Middle East. But it may be not far off to when another source of tremendous potential for the traffic through the Canal will come, when the growing industrialization of the Southeastern Asian and Far Eastern Countries increases the volume of exchange from Europe. This is another challenge to the Canal. However, no one can discharge the fact that the natural conditions there offer constant threat of silting and deteriorating as happened repeatedly before.

Finally, the author is well credited with the encompassing capacity of technological, historical (especially Hellenism and Egytology) and biblical background, coupled with the perseverance to undertake such herculean task to go over the voluminous records. Such digestion is very helpful to any one to deal with the Canal itself or with waterways in general. It is almost certain that the paper will render terrific service in time to come.

After all, the Canal occupies a very strategical position. In the last two global wars, intense struggles were staged to keep the Canal under control and this does not count lightly in the final victories. Now, eyes are once again on this area and no one can ensure a smooth sailing, if conflict of major scale occurs under the complicated situation there. It will be of great value if the author includes an account on this point in his next paper, as he promises in this.

W. E. POTTER,[1] M. ASCE.—The parallel engineering purposes and common heritage of the Suez and Panama Canals has long been recognized. Both waterways have, as the principal mission, the safe expeditious movement of ocean-going ships from major seas to contiguous, but not naturally connected oceans. The Mediterranean was used from time immemorable and its connection to the Indian Ocean through the Red Sea by means of the Suez Canal was the germ that has sparked the tremendous economic development of the Near East. Similarly, the Panama Canal connecting the Caribbean Sea which had been a means of communication for ages between the Antilles and the continents, and the Pacific Ocean, is the latter day levening of commerce to the Middle America States and West Coast of South America.

Geologically, the parallel also obtains, for at one time at each of these strategic locations, Panama and Suez, a strait existed. Man's ingenuity was required to restore this strait. Furthermore, the first major organization and construction effort was made by the same man, Count Ferdinand de Lesseps.

Here the parallel ends; De Lesseps was successful in his construction of a sea level canal at Suez. This was strictly a commercial enterprise to which the Egyptian Government granted the right to cut, maintain and operate a maritime canal for 99 years from completion of the work, at which time the

1. Governor, Canal Zone Government, Balboa Heights, Canal Zone.

possession of the waterway was to revert to the Egyptian Government. This 99 year period has been cut short by the Nationalization Act in 1956.

De Lesseps was unsuccessful in his efforts to construct a sea level canal at Panama, having been defeated primarily by disease. The construction of the canal at Panama was achieved through the efforts of the United States Government, with the cooperation and consent of the newly created Republic of Panama. In return for a guarantee of independence and other considerations, the Republic of Panama granted to the United States in perpetuity, (a) the use, occupation and control of a Canal Zone for the construction, maintenance, operation, sanitation, and protection of a canal; (b) the exclusive right to the exercise of sovereign powers in the Canal Zone; (c) the right to the use of bodies of water within Panama's limits; and (d) a monopoly as regards canals across the Isthmus. The Hay-Pauncefote Treaty of 1901 between the United States and Great Britain, assuring the neutrality of the canal was reaffirmed. The 1903 Treaty has been supplemented by (a) the 1914 Boundary Convention, definitely fixing the boundaries of the Canal Zone, with subsequent modifications in 1920, 1924 and 1931; (b) the 1936 Treaty and (c) the 1955 Treaty and Memorandum.

The 1936 Treaty covered the following:

(a) The two Governments declared, in view of the official opening of the Canal, that the provisions of the 1903 Convention contemplate the use, occupation and control by the United States of the Canal Zone for the purposes of the efficient maintenance, operation, sanitation, and protection of the Canal and of its auxiliary works.
(b) The United States undertook to continue the maintenance of the Canal for the encouragement and use of inter-oceanic commerce; and the two Governments declared their willingness to cooperate to insure the full enjoyment of the benefits which the Canal should afford the two nations that made possible its construction as well as all nations interested in world trade.
(c) By an exchange of notes at the same time it was agreed that the word "maintenance," as applied to the Canal, should be construed as permitting expansion and new construction when these are undertaken by the United States in accordance with the said Treaty.
(d) The two Governments agreed that, in the event, then deemed improbable, further lands and waters should be in fact necessary for the Canal, or its protection, such Governments would "agree" upon the necessary measures.
(e) The annuity paid Panama was redefined at $430,000. Various other provisions had the effect of restricting the patronage of Canal Zone sales and service facilities to those connected with the enterprise.

The 1955 Treaty and Memorandum covered:

(a) The annuity to Panama was increased to $1,930,000.
(b) The United States' railroad and highway monopolies across the Isthmus were relinquished, subject to a restrictive agreement relative to the maintenance of an inter-oceanic railroad or highway within Panama by a third country or nationals thereof.
(c) The boundary line between Colon and the Canal Zone was redefined so as to remove from the Canal Zone and place in Colon a strip of water around the northerly, easterly, and southerly shores of Colon.

(d) In the personnel field, the United States agreed to seek (and Congress has since enacted Act July 25, 1958, P.L. 85-550, 72 Stat. 405) legislation to establish a single basic wage for Canal Zone agencies, as well as extension of the Civil Service Retirement Act to Panamanians; and agreed to afford equality of opportunity to Panamanians.
(e) Various measures were adopted to afford greater opportunity to Panamanian commerce to obtain Canal Zone business.

Thus, there is little or no parallel between the Suez and Panama enterprises existing in the political field.

Physically, the existing canals differ widely; the Suez is a sea level canal through an extremely arid region where blowing sand is a major operating problem; Panama is a lock canal through an extremely humid region in which driving rain and fog is, at times, a serious problem. Lock operation and maintenance problems prevail only at Panama; the size of the existing locks limits the absolute maximum size of ship which can transit Panama to approximately 60,000 tons dead-weight and a maximum draft of 38 feet in fresh water. At Suez the maximum ships passed up to 1955, as reported in Dr. Shu-t'ien Li's paper, is 46,500 tons, gross, with a draft of 34.5 feet.

Dr. Shu-t'ien Li has produced an intriguing chronicle, and an invaluable bibliography to the approximately 106 items of published works, records, and literature relating to the Suez Canal. The relative paucity of such items, records and literature, and the need to document and list those which are available is recognized, when it is known that the Canal Zone Government Library catalogs approximately 7,000 items on the Republic of Panama and the Panama Canal. The author refers to a later paper he proposes, which will cover aspects of the Suez Canal since nationalization in 1956. We shall look forward with considerable interest to this up-dating of Dr. Shu-t'ien Li's chronicle.

ARTHUR B. CLEAVES,[1] M. ASCE.—This is a most interesting paper on the development and engineering events since historical times, of the Suez Canal.

One would wish that there might be a sequel stressing the basic types of geological and soils engineering studies made. The varied types of revetments utilized are intriguing and one wonders how many repeated revetment failures have occurred? Was it found that the usage of nummulitic quarry limestone was short-lived because of weak natural consolidation, or "bonding" of the discrete particles? Did the concrete block revetments fail because of low shearing strengths in the soils of the canal slopes? Are the "slip-out" slides confined to specific reaches of the canal, and if so what studies have been made in trying to effect their control?

In view of the fact that such slides appear to be common; the accumulation of wind-blown silts a constant nuisance; and possibly the ideal type of revetment still an unknown factor, a modern geological and soils engineering investigation might be in order. The cost of such a study would be a very minor portion of the annual canal maintenance and its results could be most rewarding. The geological studies might well utilize seismic and electrical resistivity (geophysical) techniques; petrographic examinations; clay technological

1. Prof. Geological Eng., Washington Univ., St. Louis, Mo.

approaches through X-ray and geothermal examinations; and the usual soils mechanics investigations. It is believed that modern technological studies following these lines could open the door to genuine maintenance savings.

SCIENTIFIC DEVELOPMENTS IN RIVER TRANSPORTATION[a]

Discussion by Shu-T'ien Li

SHU-T'IEN LI,[1] M. ASCE.—The author has presented some very valuable results on tow resistance, delivered horsepower, push, and tow efficiency versus speed; and costs versus number of barges. It is beyond the per adventure of a doubt that this effort and expense are eminently justified by whomsoever that operates and lives from river equipment so long as perfection has not yet been achieved.

When the writer was promoting the First National River Hydraulic Laboratory of China near the end of the 1920's, he had a chat with the late John R. Freeman, Hon. M. ASCE, during the International Engineering Congress held in Tokyo in 1929. Dr. Freeman was then advocating river hydraulic laboratories in the United States. Both his advocacy and the writer's were after German patterns of this kind of hydraulic laboratories. Dr. Freeman was so enthusiastic and positive that he told the writer: "There is nothing else that will yield a thousand or more per cent in dividend than the river hydraulic laboratory."

Three decades have since passed. There has been a tremendous change in the approach toward "scientific developments", using the author's phrase, in almost any industrial endeavor. Pure "empiricism" has been replaced by semi-mathematical and theoretical "rationalism" plus experimental verification well planned in advance. This new approach, beginning with mathematical and theoretical deductions, thereby organizing all physical parameters involved into logical formulations, is conducive to the saving of time and expense, affords more precise predictions as to how a dependent variable would vary as a function of the independent(s), and lends practical convenience in the application of results to similar situations with varying parameters.

In order to keep this discussion as brief as possible, the writer will be concerned only with "tow resistance as a function of speed" for which the author has given eight graphs representing his experimental results.

The writer values these results as much as the author does, and hence this discussion. The pertinent inquiry arises: Is speed the only independent variable of tow resistance? How would the results of one set of experiments for one group of towboat and barge formation in their physical measurements as a prototype reduced in model scale, apply to another group with different formation and different physical parameters? If all the physical parameters involved were first logically and theoretically organized into a formulation by the application of basic mathematical and hydrodynamical principles, it would

a. Proc. Paper 1772, September, 1958, by Clarence Reuben Horton.

1. President, Chinese Section, Permanent International Association of Navigation Congresses; Formerly Vice Chairman of the Board, First National River Hydraulic Laboratory of China; Cons. Engr., Mobile, Alabama.

become possible to transform the results of one set of prototype–model experiments to predict the performance of another prototype.

The Early Newtonian Conception

The conception, dating back to Sir Isaac Newton (1642-1717), states: The resistance with which a fluid opposes the motion of a body immersed in it, by virtue of its inertia, must be proportional to the area (A) of the section of the body at right angles to the direction of motion, and also proportional to the density (ρ) of the fluid, and to the square of the velocity (V). This conception could be mathematically demonstrated by the argumentation that

$$FT = MV$$
$$F(1^{sec}) = (\rho AV)V = \rho AV^2 \qquad (1)$$

Newton regarded the fluid medium as consisting of particles free to move but at rest, which are regularly reflected by the moving body. This theory implied that the laws of collision of elastic bodies are applied to the resistance of a fluid. Eq. (1) has proved untenable under the light of later developments of hydrodynamics.

Hydrodynamical Versus Newtonian Conception

During the twentieth century, perhaps no other single fluid dynamicist has contributed more to the science of fluid dynamics than Ludwig Prandtl, formerly Professor of the University of Göttingen and Director of the Kaiser Wilhelm Institut für Strömungsforschung. Prandtl states:[9,15] "The Newtonian conception of fluid resistance has been replaced by the hydrodynamical conception, according to which the resistance consists of the pressure differences and frictional stresses arising from the fluid flowing round the body."

Fundamental differences between the Newtonian conception and the hydrodynamical conception, when applied to a barge tow, are as follows:

(1) In the former only the geometrical configuration of the bow is considered, whereas the phenomena giving rise to resistance are also pronouncedly affected by the configuration of the stern due to the formation of the wake in the rear.

(2) In the former the resistance of an immersed barge tow is obtained by simple summation over all the elements of surface at right angles to the direction of motion, whereas from the hydrodynamical point of view this is inadmissible. Classical experiments[4] of the French Engineer, Alexandre Gustave Eiffel (1832-1923), and logical illustrations by Prandtl,[9,15] both confirm the difference. It may then be inferred that two bows of the same configuration and same vertical projection will have different total resistance than one bow of like configuration and twice the vertical projection.

(3) A longer barge or a longer barge train will have a relatively lower resistance than the shorter, because the system of eddies is reduced in extent in the former owing to the flow re-attaching itself to the side surface of the hull, so that the suction on the rear of the barge is less than in the shorter case. Again experiments[4] of Eiffel and Modell versuchsanstalt at Göttingen in an artifical current of air under the direction of Prandtl,[9,15] both confirm the statement. This constitutes

one of the fundamental explanations of why the author's graphs show less costs as the number of barges increases while the towboat power increases.

The Hydrodynamical Law of Resistance

As Prandtl states, the resistance is produced by "pressure differences" and "frictional stresses." In general, the former predominate and may be taken as proportional to the dynamical pressure (p_d) corresponding to the velocity (V). When a barge moves through water flowing at velocity (V) relative to the barge, the water is dammed up immediately in front of the barge and divides in all directions in order to flow round it. At the center of the dammed-up region, the "stagnation point," the water comes entirely to rest. Thus if p_s is the pressure at the stagnation point and p_o the pressure at the same depth in the undisturbed water, by applying Bernoulli's theorem to the stream-line between a fore undisturbed point and the stagnation point, we have

$$\frac{p_s}{w} + 0 = \frac{p_o}{w} + \frac{V^2}{2g}$$

$$As\ w = \rho g, \qquad p_s - p_o = \frac{1}{2}\rho V^2$$

The excess pressure $p_s - p_o$ is physically due to the conversion of velocity energy into pressure energy, and may be called the "dynamic pressure", or "kinetic pressure", or "stagnation pressure." Hence,

$$P_d = \frac{1}{2}\rho V^2 \tag{2}$$

which is the dynamic pressure on a unit area of the bow if it were at right angles to the direction of motion. When any elementary area (dA) makes an angle (θ) with the longitudinal direction, then the effective elementary area is $\sin\theta\, dA$, and the total dynamic pressure (P_d) acting on the bow is given by

$$P_d = \frac{1}{2}\rho V^2 \int^S c \sin\theta\, dA \tag{3}$$

where both c (a constant for a given surface in space c) and θ may vary at different elementary areas as a function of the configuration of the bow, and $\int^S$ denotes the integral over the surface of the bow. For a given shape of the bow with a definite set of physical dimensions, we may write

$$P_d = \frac{1}{2} C \rho V^2 \int^S \sin\theta\, dA \tag{4}$$

$$and \qquad Kbd = \int^S \sin\theta\, dA \tag{5}$$

where b and d respectively = the width and draft of one barge, and K a coefficient. Thus,

$$P_d = \frac{1}{2} CKbd\rho V^2 \tag{6}$$

For refined work, "C" should be expressed as a function of the Reynolds Number, such that

$$C = f(N_R) = f\left(\frac{Vl\rho}{\mu}\right) = f\left(\frac{Vl}{\nu}\right) \tag{7}$$

where μ is "coefficient of viscosity," and ν the "kinematic viscosity."

The total frictional stress, or tangential resistance (T), all over the immersed hull of a barge, if τ represents the unit tangential resistance over an elementary area (dA), may be evaluated from the equation:

$$T = \int^S \tau \cos\theta \, dA \tag{8}$$

Where $\int^S$ designates "integral over the entire immersed surface of the hull of a barge or a barge train." When τ is fairly constant, we may write

$$T = \tau \int^S \cos\theta \, dA \tag{9}$$

When T is an insignificant quantity, we may throw its effect into "C" of Eq. (6).

In addition, the difference in hydrostatic pressure (P_h) between the bow and the stern must be considered. If there is a difference in water elevation (z) at the bow and at the stern due to slope of water surface and damming-up of water at the bow, and "s" is the net slope of water between the bow and the stern for one barge of length (ℓ), then

$$z = \ell s \tag{10}$$

and the hydrostatic pressures at the bow ($P_{h,b}$) and at the stern ($P_{h,s}$) are given by:

$$P_{h,b} = \tfrac{1}{2} w b d^2$$

$$P_{h,s} = \tfrac{1}{2} w b (d-z)^2$$

whence the net difference

$$P_h = \tfrac{1}{2} w b (2dz - z^2)$$

As "z" is very small, the term z^2 may be neglected, and

$$P_h = wbdz = wbd\ell s \tag{11}$$

Because the displacement (D) of a barge is nearly given by

$$D = wbd\ell \tag{12}$$

hence

$$P_h = Ds \tag{13}$$

Summing up the dynamical pressure, frictional resistance, and hydrostatic pressure difference, we get the resistance (R) of a tow with a single barge as

$$R = P_d + T + P_h \tag{14}$$

By substituting values from Eqs. (6), (8), and (13), we have

$$R=\tfrac{1}{2}CKbd\rho V^2+\int^S \tau\cos\theta\, dA+Ds \qquad (15)$$

When there are "i" barges in abreast and "j" barges in line, where "i" may be from 1 to 5, and "j" from 1 to 6 or more, then the total resistance of the barge formation of N = ij barges is given by

$$R=\tfrac{1}{2}C_{ij}K_{ij}\,ibd\rho V^2+\int^S \tau\cos\theta\, dA+NDs \qquad (16)$$

The component resistances in Eq. (16) may also be obtained from purely dimensional analysis by using Vaschy-Buckingham Theorems[3,5] and applying the writer's corollaries and simplified method.[17] The writer obtained similar equations as (15) and (16) in connection with an investigation cited under reference No. 18.

Introduction of the Froude Number

The law of comparison between ship prototype and model was first discovered by William Froude of England in 1870 in connection with his pioneering experiments in the study of resistance to motion of ships by the use of models. This classical law of similitude for ship models states that the ratio of the inertia force (Ma) to the gravity force (W) in model and prototype are equal. If "L" and "T" respectively denote length and time, then

$$\frac{\text{Inertia Force}\,(F_i)}{\text{Gravity Force}\,(F_g)}=\frac{Ma}{W}=\frac{\rho L^3\left(\frac{L}{T^2}\right)}{\rho g L^3}=\frac{L^2V^2}{gL^3}$$

whence the dimensionless parameter of Froude Number (N_F) is given by

$$N_F=\frac{F_i}{F_g}=\frac{V^2}{gL} \qquad (17)$$

But Eq. (16) may be written thus,

$$R=\tfrac{1}{2}C_{ij}K_{ij}N\frac{(wbd\ell)V^2}{g(j\ell)}+\int^S \tau\cos\theta\, dA+NDs \text{ ----(16)'}$$

As $wbd\ell=D$, and $\frac{V^2}{g(j\ell)}=N_F$, Eq. (16)' becomes

$$R=\tfrac{1}{2}C_{ij}K_{ij}NDN_F+\int^S \tau\cos\theta\, dA+NDs \text{ ------(18)}$$

For a given barge formation of a definite set of dimensions and configuration, if "S" is the total immersed surface in the longitudianal direction, and "T_{ij}" the average tangential resistance in the longitudinal direction for "i" x j" barges, Eq. (18) becomes

$$R=\tfrac{1}{2}C_{ij}K_{ij}NDN_F+T_{ij}S+NDs \qquad (19)$$

where C_{ij}, K_{ij}, T_{ij} will have different values for variable "i" barges abreast and "j" barges in each line. Similar equations were deduced by the writer in connection with his investigation cited under reference No. 18.

Further Discussion of Dynamical Similarity

In ship- or barge-model testing, the water suffers inappreciable compression during flow, and consequently the elasticity forces may be disregarded. Unless the models for testing are extremely small, surface tension may also be neglected. Thus the forces left to be considered are pressure (F_p), inertia (F_i), viscosity (F_v), and gravity (F_g). If subscripts "p" and "m" denote respectively "prototype" and "model," and disregarding the hydrostatic pressure force, there results

$$\left(\frac{F_i}{F_g}\right)_p = \left(\frac{F_i}{F_g}\right)_m = (N_F)_p = (N_F)_m$$

$$\left(\frac{F_i}{F_v}\right)_p = \left(\frac{F_i}{F_v}\right)_m = (N_R)_p = (N_R)_m$$

whence

$$\left(\frac{V^2}{lg}\right)_p = \left(\frac{V^2}{lg}\right)_m \tag{20}$$

$$\left(\frac{Vl\rho}{\mu}\right)_p = \left(\frac{Vl\rho}{\mu}\right)_m \tag{21}$$

The solution of these two simultaneous equations gives

$$\frac{V_p}{V_m} = \left(\frac{l_p}{l_m}\right)^{\frac{3}{2}} \tag{22}$$

which indicates that the selection of the model scale dictates the kinematic viscosity of the fluid for the towing basin. Either an appropriate fluid for the towing basin must be economically found, or if water is used in the towing basin the model scale must be unity which means full-scale model. This very fact has presented an insurmountable obstacle to perfect "scientific" dynamical similarity for ship and barge models. An expedient compromise is to obtain "imperfect dynamic similarity" by using the same Froude Numbers in model and prototype, and adjusting the test results by experimental data dependent upon the Reynolds Number. This expedient approach is permissible since the flow conditions resulting from inertia-viscosity are small as compared with those resulting from inertia-gravity.

Correlation with the Author's Experiments

As the author's paper does not contain pertinent information for evaluating the second and third terms on the right-hand side of Eq. (19), and these terms may be small as compared with the first term, let the resistance be taken as

$$R = \frac{1}{2} C_{ij} K_{ij} N D N_F + \cdots \tag{19'}$$

When V is expressed in miles per hr,

$$N_F = \left[\frac{5280}{(60)^2}\right]^2 \frac{V^2}{g(jl)} = 0.0668 \frac{V^2}{jl} \tag{23}$$

on the basis of g = 32.2 ft/sec.^{2}

The barges used in the tank tests presented on Charts 1 and 2 of the author's paper, according to a communication of W. W. Hamilton, Marine Design Section, Engineering Department, Engineering Works Division, Dravo Corporation, are of the following dimensions:

Length - 195'	Draft - 9' (fresh water)
Width - 35'	Displacement - 1,767 short tons per barge

The barges were placed three abreast.

Therefore, for 9-barge tow without dynanometer barge (lower graph, Chart 2 of the author's paper),

$$ND = 9(1{,}767)(2{,}000) = 31{,}806\,(10)^3 \text{ lb}$$

$$jl = 3(195) = 585 \text{ ft}$$

$$\frac{NDN_F}{2} = \frac{66.8(31{,}806)V^2}{2(585)} = 1{,}816\,V^2$$

and

$$R = C_{3,3}K_{3,3}(1{,}816)V^2 \qquad (24)$$

where R is in lb., and V in miles per hr.

According to Chart 2, lower graph, of the author's paper, simultaneous speed-resistance magnitudes are:

Speed in MPH	5	6	7	8	9	10
Resistance in lb	14,000	20,100	27,200	35,700	46,600	59,800
$1{,}816V^2$ gives	45,400	65,400	89,000	116,200	147,100	181,600
Hence $C_{3,3}K_{3,3}$	0.308	0.307	0.306	0.307	0.317	0.329

Whence it is evident that fairly constant values for $C_{3,3}K_{3,3}$ exist for speeds from 5 to 8 miles per hour. The mean value for $C_{3,3}K_{3,3}$ within this range is 0.307. Hence, for $5 \leq V \leq 8$ miles per hour,

$$R = 1{,}816\,(0.307)V^2$$

or

$$R = 558\,V^2 \qquad (25)$$

And unless the second and third terms of Eq. (19) are taken into consideration, for $8 \leq V \leq 10$ miles per hour,

$$C_{3,3}K_{3,3} = 0.307 + (V-8)(0.011)$$

$$C_{3,3}K_{3,3} = 0.219 + 0.011V \qquad (26)$$

whence $$R = 1{,}816(0.219 + 0.011V)V^2$$

$$R = 398V^2 + 20V^3$$

$$R = 20V^2(19.9 + V) \qquad (27)$$

It is because the tangential resistance becomes larger at higher speeds and can no longer be neglected that Eq. (25) gives slightly lower resistance for speeds greater than 8 miles per hr.

Concluding Remarks

In these days of increasing pace in the improvement of inland navigable waterways, "scientific developments" of river transportation should receive due attention among engineers engaged in competitive transportation. The author is to be highly commended for presenting his interim results to the profession, and the writer looks forward to read the results of his major testing program now being continued in a new model basin in Holland.

To restrict the length of this discussion, the writer will not discuss other interesting features contained in the author's paper. It was due to the author's emphasis on "scientific developments" that this discussion has been written to explore the basic general scientific relation that expresses tow resistance as a function of displacement, arrangement and number of barges, Froude Number, frictional stress, total immersed surface area, and water slope. The writer believes Eq. (19) represents a semi-rational mathematical and hydrodynamical foundation for evaluating tow resistance. Further, he wishes to emphasize six more points:

(1) In Eq. (19), "D", "S", "K_{ij}" can be computed for a given barge formation, configuration, and draft.

(2) In actual prototype running tests, push or pull can be calculated from the performance of the towboat engine, $N_F = \frac{V^2}{(jl)g}$ can be computed after observing the actual velocity; and "s" in the third term can be observed.

(3) In model tests, "R" and "s" can be measured; velocity is usually obtained from measurements of the dynamic pressure, and hence the latter, and not the velocity, is the original experimental quantity; and further the adjustment of test results by experimental data dependent upon the Reynolds Number is necessary for the "scientific development" the author is advocating.

(4) Having computed, or observed, or measured all the above mentioned factors in Eq. (19), and after running at least 2 tests at 2 different speeds, "C_{ij}" and "T_{ij}" can be calculated by solving two simultaneous equations. As more data become available, either method of least squares or method of regression may be applied to get the best fit.

(5) Scientifically speaking, in problems of this category, we are more interested to learn through experimentation the magnitudes of the coefficients and how they may vary with different barge formations and with different bow and stern configurations.

(6) The analysis of already obtained experimental data and the design of forthcoming model tests would enhance wider application if the basic parameters be mathematically formulated and coefficients dynamically determined accordingly.

Though the foregoing discussion on tow resistance has covered all the conceivable parameters among and immediately around the barges, and has even included the effect of fleet arrangement (or barge-train formation) as reflected by C_{ij}, K_{ij}, and T_{ij}, there exist other factors that must be given consideration in analyzing the overall towing efficiency. They are:

(a) Ratio of breadth of waterway to breadth of barge formation
(b) Ratio of depth of waterway to draft of barges
(c) Ratio of waterway cross-section to barge-formation cross-section
(d) Form of waterway cross-section
(e) Number of barges loaded, partially loaded, and empty in a formation
(f) Method of navigation
(g) Type of towboat
(h) Power of towboat
(i) Type of engine
(j) Kind of fuel
(k) Fuel utilization

to say nothing about terminal effects, and those factors other than purely physical in character.

The subject of "Scientific Developments in River Transportation" is indeed a broad one.

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NAVIGATION OF THE COLUMBIA RIVER[a]

Discussion by Robert E. Hickson

ROBERT E. HICKSON,[1] M. ASCE.—This paper presents, in a comprehensive and interesting manner, the historical development of this great river for navigation, and the resulting great increases in commerce.

There are some aspects of the work done which, it is thought additional information and observations will serve to clarify, particularly as to the lay-out and construction of channel control works. Much of the dike and dam construction work in the Willamette River in the early days (1875-1883) to which Mr. Holmes refers was promoted by local interests with assistance from the United States. Apparently it was thought at that time that closure of the side channels (Willamette Slough and west channel at Swan Island, and narrowing at some other places) would sufficiently increase velocities in the river to reduce shoaling.

These old works in the Willamette were, however, largely ineffective, principally for the reason that flow of the Willamette and its velocities are not high enough to maintain the section necessary to accommodate ocean shipping in the harbor.

At Swan Island, the ship channel formerly was on the east side of the island. It was narrow and had a bad turn at the lower end. Portland's first air field was built on Swan Island, for which the island was raised and a dredge fill causeway constructed across the upper end of the east channel. The west (wide) channel was dredged from a depth of about 8 feet at that time, and the old closing dike, previously referred to, was removed. The dike and dam at upper end of Willamette Slough, and the longitudinal Post Office Bar dike were also removed about that time.

Dredging has always been depended upon for maintenance of the channel in the lower Willamette (Portland harbor), as it necessarily will be in the future.

The planning and layout of the modern system of control dikes for the lower Columbia were based on a study of the needs for control at points where the natural river section was too wide, and shoaling occurred. Wide reaches were narrowed to the width found at natural sections of equilibrium where depths were as desired. There are many such sections to be found in the natural river channel.

The works thus constructed under the 30' and 35' projects in the Columbia are considered to have been very successful, as indicated by Mr. Holmes. About the same amount of dredging is now being done for a 35' x 500' channel as was formerly done on the older channel, 30' x 300', but the project now

a. Proc. Paper 1789, September, 1958, by Ray E. Holmes.

1. U. S. Corps of Engrs., Retired, Portland, Ore.; formerly member of Committee on Tidal Hydrs.

includes the Portland harbor, which was formerly maintained by the Port of Portland, as well as a deeper channel to Vancouver, Washington, five miles above the mouth of the Willamette.

As stated in paper 1789, the dikes on lower Columbia are of the permeable type. This type lends itself well to channel improvement and maintenance. The dikes are constructed to about 1/2 the flood stage, i.e. from a minimum of 6' in the lower reaches to 10' or 12' above low water farther up stream. No violent disturbance is caused and channel improvement is gradual. The channel depths on bars have been gradually increasing for 30 to 40 years and are now approaching a condition of equilibrium. Dredge deposits are placed between dikes, and when a growth of brush or willows starts, suspended river silts during floods soon build up these areas to natural bank elevations. At such locations dredging has been reduced to small amounts and is now to a large extent in over-depth.

Reference to the article in The Military Engineer, Vol. 22, May-June, 1930, page 217, will show earlier data on time and nature of shoaling in the lower Columbia River before control works were completed.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbol (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1059 is identified as 1059 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(ST2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(HY2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

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JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA2), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA2), 1682(SA3), 1683(PO2), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(SA5), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c], 1833 (HY5)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 (SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(ST2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 (CO1), 1959(CO1).

MARCH: 1960(WW3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA3), 1968 (SA3), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977 (WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA3), 1985(SA3)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

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c Discussion of several papers grouped by divisions.

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Your attention is invited

NEWS OF THE WATERWAYS AND HARBORS DIVISION OF ASCE

JOURNAL OF THE WATERWAYS AND HARBORS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

March, 1959

PRINCETON CONFERENCE—BERTHING AND CARGO HANDLING IN EXPOSED LOCATIONS

A conference on Berthing and Cargo Handling in Exposed Locations was held at Princeton University on October 20-21, 1958 in connection with the Department of Civil Engineering's River and Harbor Program. The subject of the conference was suggested by the fact that some of the huge tankers and bulk cargo carriers recently built and now under construction are too large to reach existing berths in protected waters at many ports of call. The cost of the channel improvements required to make existing piers and wharves accessible to these super-ships, some of which are larger than the QUEEN MARY and QUEEN ELIZABETH, might in many cases be so great as to be considered prohibitive. A study of the feasibility of berthing, loading and unloading such vessels in relatively unprotected waters is therefore of great importance.

Furthermore, it was felt that any study of the response of ships to wave action would yield information important to persons dealing with the development of "roll-on, roll-off" and allied methods of loading and unloading cargo vessels of normal size in both protected and unprotected waters, as many such methods require a degree of fixity between the ship and the pier or wharf that is not demanded by conventional cargo handling through hatches.

The conference lasted two days. Eight papers were presented on subjects and by speakers as listed below:

The Energy Problem

Basil W. Wilson, Associate Professor of Oceanography and Meteorology, Agricultural and Mechanical College of Texas—The response of a ship to waves and methods of estimating the total kinetic energy which must be absorbed by permissible deformations of the ship, the fender system and the pier.

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Energy Absorption by the Ship

Edward V. Lewis, Head of the Ship Division, Stevens Experimental Towing Tank, and Sidney F. Borg, Head of Civil Engineering, Stevens Institute of Technology—Considerations leading to the berthing of ships in unprotected waters. Ship structural design from the point of view of energy absorption and the avoidance of berthing damage.

Elastic Fender Systems

Zusse Levinton, Tippetts-Abbett-McCarthy-Stratton—The energy absorbing capabilities of fender systems which absorb energy by non-destructive elastic deformation of specially introduced elastic members, such as springs or rubber cushions.

Gravity Fender Systems

A. L. L. Baker, Professor of Concrete Technology, University of London—The energy absorbing capabilities of fender systems which absorb energy either by lifting the entire fender system or by lifting heavy components provided for energy absorption.

Energy Absorption by the Pier or Wharf Structure

E. H. Praeger, Praeger-Kavanagh—Methods of estimating the capacity of the pier or wharf to absorb energy without suffering damage, and the effect on pier design of energy absorption considerations.

The Sea Berth

Captain R. M. Stall, California Shipping Company—Loading and unloading super-tankers at sea berths by submarine pipe lines—no pier or wharf.

The Ship Handling Problem

B. W. Havens, Socony Mobil Oil Company—The feasibility of bringing ships alongside a pier or wharf, or mooring them at sea berths, in exposed locations.

Offshore Mooring Facilities for Tankers up to 100,000 Dead Weight Ton Capacity

F. H. Stracke, Esso Research and Engineering Company—Discussion of associated design problems.

A discussion period followed the presentation of each paper. The value of the conference was greatly enhanced by the effectiveness of the audience participation in these discussion periods. The conference was attended by 172 delegates from industrial firms, offices of consulting engineers, contractors, and the Governments of the United States and Canada.

Informal luncheons were held in the main University dining hall on both days of the conference and on the night of October 21st there was a dinner at which President Emeritus Harold W. Dodds spoke most effectively on some of the problems of higher education.

This was the third and also the largest and best attended conference on some phase of river and harbor engineering which has been held at Princeton.

The first conference was held in October 1956 on the subject of Cofferdams. The 1957 conference was on The Design of Bulkheads and Quay Walls. The next conference which is tentatively scheduled for January 1960 will be on Navigation Problems of the St. Lawrence Seaway and the Great Lakes.

CHANGES ON DIVISION RESEARCH COMMITTEE

Col. Edmund H. Lang, former Secretary of the Waterways and Harbors Division Research Committee, has been appointed Chairman of the Committee. Col. Lang is the Director of the U. S. Army Engineer Waterways Experiment Station at Vicksburg, Mississippi. Prior to his assignment at Vicksburg, Col. Lang served as District Engineer, U. S. Lake Survey, Detroit.

Col. Lang replaces as Chairman of the Research Committee Gen. Herbert D. Vogel, Chairman of the Tennessee Valley Authority, who has withdrawn from the Committee due to other obligations.

Mr. Joseph M. Caldwell of the Beach Erosion Board has been appointed a member of the Research Committee replacing Prof. J. W. Johnson who is a member of the Division Executive Committee.

COMMITTEE ON REGULATION AND STABILIZATION OF RIVERS CONSIDERING SEVERAL INVESTIGATIONS

At the November 13-14, 1958 meeting of the Division Committee on Regulation and Stabilization of Rivers, the need for special studies on several subjects was considered. These subjects were:

a) Considerations which determine whether navigation is to be attained by open channel methods or by locks and dams.
b) Discharge and sediment relations in rivers
c) Alignment as it affects open-channel methods of improvements
d) Soil characteristics of bed and banks as they affect open-channel improvements
e) Contraction works and bank stabilization
f) Dredging

The discussions developed that of the several subjects, probably "Bank Stabilization" appeared to be the most urgent. In considering the scope of a task force on this or other subjects, the view was developed, endorsed, and emphasized by Col. Lawrence B. Feagin, Chairman of the Division Executive Committee, that any task force should have a very definite objective, practical of accomplishment of the task within a reasonable period of time. Dr. Lorenz G. Straub suggested that a task force compile data to reflect clearly "Current Practices in the Stabilization of Alluvial Rivers," to include not only "what" is being done, but also "why", to include factors which determine design as well as experienced data relating to performance. To cover the practices rather broadly, it was suggested that there be considered those on the principal alluvial rivers in the country: Mississippi, Missouri, Arkansas, Rio Grande, Columbia, Colorado, and Sacramento; and to include for each river data on the closely related factors of the sediment transport, bank and bed materials, slope and discharge, and cross sections.

Through arrangements made by Col. Feagin and Mr. R. W. Sauer and courtesies extended by the Lower Mississippi Valley Division and Vicksburg

District, Corps of Engineers, the Committee observed the Mississippi River from Mayersville to Vicksburg by boat. The trip permitted the members to see and discuss at first hand the work being performed and problems encountered in development of the bank stabilization program of the lower Mississippi River to aid in obtaining a channel of project depth as well as fixing the river's alignment. As a part of the trip, a stop was made at the articulated concrete mattress sinking plant, which reflected the excellent organization of operations by the District. The trip was especially helpful to the Committee in emphasizing the importance of its assignment.

The reorganized Committee on Regulation and Stabilization of Rivers is under the chairmanship of J. F. Freidkin of the International Boundary and Water Commission, U. S. and Mexico. The other committee members are Dr. Straub, Director, St. Anthony Falls Hydrological Laboratory, Raymond H. Haas, Clement P. Lindner and Raymond L. Huber.

MARYLAND SECTION CONTINUES ACTIVITY

Since 1954, the Waterways and Harbors Committee of the Maryland Section has sponsored an annual meeting concerning waterway development. At the January 14 meeting of the Maryland Section, Colonel Stanley T. B. Johnson, District Engineer, U. S. Army Engineer District, Baltimore, spoke on the work of the Corps of Engineers, in general, and of the Baltimore District, in particular. The major part of his talk dealt with navigation improvement and flood control activities. John T. Starr is the Local Section Representative—(Maryland) of the Committee on Cooperation with Local Sections.

NAVIGATION AND FLOOD CONTROL COMMITTEE ADVANCES PLANS FOR FUTURE SESSIONS

The Waterways and Harbors Division Committee on Navigation and Flood Control Facilities continued its planning for convention papers at the meeting held during the New York Convention. The Committee plans to present a paper on flood control at the Los Angeles Convention; papers on lock and dam designs and Great Lakes navigation and economic potential at the Cleveland Convention; papers on flood control structures, the St. Lawrence Seaway, and rehandling of dredged material at the Washington Convention; and papers on lock and dam construction, dredging, model study, management and operation of high dams, tidal power, hurricane barriers, and other flood control and navigation subjects at the New Orleans, Reno and Boston Conventions.

The Committee has expanded its membership by including:

Colonel Arthur C. Nauman, Assistant Chief of Civil Works for Middle-western Divisions, in the Office of Chief of Engineers;

Mr. H. O. Ogrosky, Chief, Engineering Division, Soil Conservation Service; and

Mr. James S. King, Civil Engineer, U. S. Army Corps of Engineers, North Central Division.

Col. Nauman replaces Mr. Almern F. Griffin, who resigned due to other obligations.

LOCAL SECTION ACTIVITIES

The Committee on Cooperation with Local Sections is planning to compile a list of the papers appearing in the Division Journal and of the Civil Engineering articles based on presentations at Division Sessions over the last seven years to illustrate to non-Division members the scope of the Waterways and Harbors Division. The Committee plans also to keep in close contact with Local Sections to develop additional interest in the Division. Francis G. Christian is Chairman of the Committee.

PUBLICATIONS OF INTEREST TO DIVISION MEMBERS

The American Geophysical Union is combining the scientific and technological material in its bimonthly TRANSACTIONS with the quarterly JOURNAL OF GEOPHYSICAL RESEARCH as a monthly starting in 1959. This will contain much material bearing on Hydrology of interest to our members. The subscription rate for the new JOURNAL is $16.00 for the calendar year. Memberships, however, are invited. Membership is $10.00 per calendar year and includes a subscription to the JOURNAL. Any who are interested should write to the office of the American Geophysical Union, 1515 Massachusetts Avenue, N. W., Washington 5, D. C.

HYDRAULIC RESEARCH IN THE UNITED STATES 1958 is the latest in a series of annual publications dating back to 1951 and contains reports on hydraulic research conducted within the past year in the U. S. and Canada. The information is compiled from reports by various hydraulic and hydrologic laboratories in both countries. National Bureau of Standards Miscellaneous Publication 224, Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., $1.25.

Cargo-handling techniques have not kept up with technological advances in propulsion and other aspects of ship design, according to the 40-page study, SURVEY AND ANALYSIS OF CARGO CONTAINERS FOR CARGO SHIPS of the Maritime Administration. Copies of the study (PB 151239) may be purchased from the Office of Technical Services, U. S. Department of Commerce, Washington 25, D. C., at $1.00 per copy.

U. S. LAKE SURVEY INAUGURATES NEW LAKE LEVEL FORECAST BULLETIN

Great Lakes levels and the forecast of these levels by the U. S. Lake Survey of the Army Corps of Engineers are becoming more and more important with the nearing completion of the St. Lawrence Seaway and the excellent progress being made by the Engineers on the Connecting Channels Project. Lake levels fluctuate continually. There is a pronounced and predictable annual cycle and an unpredictable, irregular long term fluctuation. Since 1952, when the lake levels were at their all time highs, the long term trend has been down.

The Lake Survey has initiated a new forecast of lake levels and a new graphical presentation of the recorded and predicted levels. Major Ira A. Hunt, Jr. is District Engineer, U. S. Lake Survey. The new forecast consists of a forecast of the levels of all the Great Lakes and Lake St. Clair each month for six months in advance. The graphical presentation of these

forecasts shows the lake level trend for the past year, the 98-year averages, the all-time highs and lows, and the low water datums used on Lake Survey Charts of the Great Lakes. The one month forecasts utilize U. S. Weather Bureau forecasts of precipitation and temperatures. The two, three, four, five and six month forecasts utilize average net basin supplies to the lake, routed to take into consideration present lake levels. These forecasts indicate the trend if average conditions prevail.

CALIFORNIA'S SMALL CRAFT HARBORS PROGRAM

In accordance with its new small craft harbors program, various cities, counties, and harbor districts of California have submitted requests for construction loans for many projects throughout the state, both along the coast and on the banks of interior lakes and rivers. The detailed planning on most of these projects is being accomplished by private engineering firms through planning loans made available by the Small Craft Harbors Commission to the public agencies sponsoring the projects. The Small Craft Harbors Commission has recommended for legislative approval a program of loans for some twenty-seven of these projects involving an estimated construction cost of $14,889,000. The legislature is expected to act on this recommendation at an early date following which the loan money will become available on submission of the required planning data and approval of plans and specifications by the State's Small Craft Harbors Division. The sum of $5,000,000 for this loan program was made available by transfer from the State's Investment Fund and another $10,000,000 was made available through vote of the California electorate on a bill authorizing the sale of bonds for this purpose.

PORT DEVELOPMENT AT HOUSTON

The Harris County-Houston Ship Channel Navigation District, on January 7, 1959, authorized the issuance of revenue bonds in the amount of $12,500,000 for the following projects:

1)	Bulk material handling plant	$3,900,000
2)	Storage warehouses	345,000
3)	Repair and general maintenance shops	330,000
4)	Rehabilitation of Wharf No. 1	413,000
5)	Modification of area between Docks 9 and 10	149,000
6)	Grain elevator improvements	662,000
7)	World Trade Center	636,000
8)	Channel Right-of-Way and spoil disposal area	1,800,000
9)	Docks 21 and 22 on North side of Turning Basin with transit sheds	3,515,000
10)	Relocation of Port Terminal Railroad	750,000
	Total	$12,500,000

In addition, the Commission took under consideration a supplemental program for consideration after the issuance of the above-named bonds and after contracts are awarded for the work listed above, as follows:

1)	Grade separations and new entrance roadways	$ 750,000
2)	Rehabilitation of Docks 2, 3, and 4	1,350,000
3)	Rehabilitation of Manchester Docks	875,000
4)	Construction of 4 additional docks	7,200,000
5)	Enlargement of bulk handling plant	2,750,000
6)	Storage warehousing	1,500,000
7)	Enlargement of grain elevator	3,000,000
8)	Additional railroad facilities	1,250,000
9)	Enlargement of executive office building	600,000
10)	Barge Terminal	725,000
	Total	$20,000,000

A favorable report has been submitted by the Corps of Engineers for the widening of the Houston Ship Channel and deepening to 40 feet. Action on this matter awaits Congressional appropriation of the necessary funds. Mr. J. G. Turney, Local Section representative for the Committee on Local Sections, is a member of the Board of Commissioners of the Harris County-Houston Ship Channel Navigation District.

WATERWAYS AND HARBORS WORK IN HAWAII

Dredging of a second entrance channel to Honolulu Harbor, under supervision of the Honolulu District, Corps of Engineers, U. S. Army, will be started this spring. The new channel will be 35 feet deep, about 400 feet wide, and about 2 miles long. Also included in the new construction will be a bascule bridge with a horizontal clearance of 250 feet across the channel.

FOR YOUR CALENDAR

May 4-8, 1959	ASCE, Cleveland Convention
October 19-23, 1959	ASCE, Washington, D. C. Convention
March 7-11, 1960	ASCE, New Orleans Convention
June 19-23, 1960	ASCE, Reno Convention
October 9-13, 1960	ASCE, Boston Convention
April 10-15, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February, 1962	ASCE, Houston Convention
May, 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

NEWSLETTER PUBLICATION

The next issue of The Waterways and Harbors Division Journal will be in June 1959. The deadline for submission of copy for that issue is April 27, 1959. If you have any material that might be usable in the Newsletter which will accompany the Journal, please send it to:

Austin E. Brant, Jr.
Editor, Waterways and Harbors Division Newsletter
Tippetts-Abbett-McCarthy-Stratton
62 West 47th Street, New York 36, New York